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THE CONTROL OF THE WESTERN MARYLAND Railway has been sold by the City of Baltimore to the Gould interests. Baltimore practically owned all of the bonds issued by the company, aggregating $4,650,785, with $3,877,180 in unpaid coupons. The price paid for these was $8,751,370. This purchase will give the Goulds a continuous line—by connecting it with the Wabash system, extending from the Atlantic Coast to Omaha; and supplementary to this will be the Denver & Rio Grande systems, also owned by the Goulds. In all about 15,000 miles of railway will thus be dominated by the Gould interests. The Western Maryland will now be connected with the West Virginia Central by a short line from Cherry Run to Cumberland. And a 35-mile link will be built to connect the West Virginia Central with the Little Kanawha road at Palestine, W. Va. It is said that contracts amounting to $10,000,000 have already been let for the construction of links to complete a standard-gage line from Baltimore to Zanesville, O., where the main line of the Wabash is reached.

THE PROGRESS OF CHINESE RAILWAYS forms part of the late report of Mr. F. E. Taylor, of the staff of the Inspector-General of Customs of China. He reports satisfactory progress in building short sections of new line and restoring line destroyed by the Boxers. On the Pekin-Hankow line the section between Lu-Kou-Chiao and Paoting-fu has been rebuilt and the line extended south to Cheng-ting-fu, 162½ miles from Pekin. It is expected that the line will be extended this year to Shun-te-fu, 77½ miles further. Northward from Hankow trains will be running by May 1 to Hsiuyang, 136½ miles, and by the end of the year it is expected the line will be nearly 200 miles north of Hankow. The Chinese railways already built are much patronized and business increases rapidly along their routes.

THE CAPE-TO-CAIRO RAILWAY, says Reuter's Agency, will build a steel bridge, of a single span of 500 ft. across the Zambesi River at Victoria Falls. The Bulawayo-Zambesi section, 275 miles long, is expected to be opened in 1903. The work of connecting the Bulawayo and Salisbury sections is progressing rapidly; and the rails are laid from Salisbury to Sabakwe, 60 miles. From the Bulawayo end, northeast, the railway has reached the Arguza River; and it is hoped that by the end of the present year trains can run from Capetown to the port of Beira, in the Mozambique Channel, by way of Bulawayo, Salisbury and Umtali.

GROOVED GIRDER RAILS have been approved by the Board of Surveys, Philadelphia, Pa., for relaying parts of the line of the Union Traction Co., of that city. They are 9 ins. high, weighing no less than 135 lbs. per yd. The head is very similar to that of the New York rail. Brick filler blocks will be laid on either side of the web, and the street paving will be of granite blocks.

STEEL COAL CARS are now standard on the Baltimore & Ohio R. R., and it is not intended to build any more wooden cars for this service. The standard car is a hopper-bottom steel car of 100,000 lbs. capacity, but for general service there is also an 80,000-lb. steel gondola car with sides 46 ins. high. These cars are built by the Pressed Steel Car Co. to the order of Mr. F. D. Casanave, General Superintendent of Motive Power of the Baltimore & Ohio R. R.

COMPOUND LOCOMOTIVES are practically not used on the Pennsylvania R. R., and many compounds acquired with roads absorbed by the Pennsylvania system have been converted into simple engines. It is found that the simple engines give greater mileage with less time and cost for repairs, while coal is obtainable at so cheap a figure that the fuel economy does not begin to compensate for the loss due to the greater time which the compounds spend in the shops. The company now has some 400 consolidation engines of 193,000 lbs. weight, and these are the heaviest of its engines, with the exception of similar engines for the mountain divisions, which weigh about 208,000 lbs.

A VESTIBULE TRAIN has been put into service on the New Lancashire & Yorkshire Railway, in England, running between London and Manchester. The train can carry 288 passengers; it is vestibuled throughout the first-class portion and the seats are there arranged on the American plan, with a continuous center aisle. It includes eight cars, but the two third-class cars carried are five-compartment cars. The train is lighted by electricity on the Stone system, and is steam heated. The signal to the guard is by means of a chain hung inside and just over the windows.

REGULATING TANKS to control the flow of dirty surface water to sewers are advocated by Mr. J. Gust Richert, of Stockholm, in the London "Surveyor" of April 18. In a system of sewers designed to remove house sewage and a minimum of storm water he would provide say one tank to each city block, with an outlet just large enough to discharge the desired maximum amount of storm water into the sewer. All surplus street drainage would flow through a properly designed surface gutter leading away from the tank inlet and discharging as in the strictly separate system of sanitary sewers. Where a separate system of both sanitary and storm sewers is in use, and it is desired to divert to the storm sewers all minimum rainfalls and the first runoff of all other storms, the regulating tank would be provided with two outlets at different levels. The lower outlet, of fixed capacity, would discharge into the sanitary sewers and the upper outlet, of capacity equal to the remainder of the surface drainage, into the storm sewers. If conditions were such that the increased head in a heavy storm resulted in too heavy a flow into the sanitary sewer, in the plan just outlined, then two chambers would be constructed, one with a constant water level and provided with an automatic shut-off valve. Still another design provides for a gradual diminution of the discharge into the sanitary sewer until, when the storm runoff reaches a certain volume, the flow to the sanitary sewer will entirely cease. The author states that the great difference between such regulating tanks and the ordinary storm overflows is that regulating tanks serve as an inlet to the sanitary sewer of a definite calculated quantity of foul rain water, while overflows discharge into the storm-water sewer an "incalculable quantity of rain water and sewage." Editorial comment on this general subject will be found elsewhere in this issue.

NO BIDS FOR THE $28,000,000 HAVANA LOAN were received on May 2. The proposed bonds were to be used to retire existing bonds, and to supply money for the provisional paving and sewerage contract awarded not long ago.

THE MOST SERIOUS RAILWAY ACCIDENT of the week was a butting collision between a west-bound immigrant train and an east-bound fast freight on the Pittsburg Division of the Baltimore & Ohio R. R. Co., near Rockwood, Pa., on May 4. The two trains crashed together at high speed on a straight piece of track; two persons were killed, two fatally hurt, and over forty others injured. There is no explanation as yet for the freight being on the track; the immigrant train was late and had been given extra time for the remainder of the single-track portion of the division.——On the afternoon of May 2 a bad wreck occurred on the New York Central & Hudson River R. R., near Clyde, N. Y. A west-bound fast mail train ran into a through freight coming in the opposite direction. An engineer and a fireman were killed and thirteen mail clerks seriously injured. The freight train had taken this track to permit an east-bound passenger train to pass, and the towerman, not noticing the movement, gave the mail a clear track, with the collision immediately resulting.

THE TEXAS OIL GUSHERS have ceased flowing, according to a dispatch to the New York "Times." A short while ago, almost at the same time as the latest earthquake disturbance in Central America, the great spouting wells ceased their activity. For a time the fact was kept secret, but it is now regarded as certain that all oil produced in the Beaumont field in the future must be pumped. It is said that since the Lucas well struck oil under high pressure and set a new mark for flowing oil wells, more than 500 "spouters" have been put in within the narrow area of Spindle Top Heights without any observable diminution of the pressure. All the wells seem to have died down simultaneously, and while the oil is still there, it must be pumped from the wells instead of flowing out under its own pressure.

THE BORATE MARSH DEPOSITS, in Harney county, Oregon, and from which the Rose Valley Borax Co., of Reno, Nev., manufactures borax, is described by W. B. Dennis in a recent number of "The Engineering and Mining Journal." The deposit covers about 10,000 acres of low land south of Lake Alvord. This area is perfectly level and treeless, and the ground is covered several inches deep with a white incrustation of borate of soda, containing sodium carbonate, sodium sulphate, sodium chloride, etc., in varying proportions. This surface deposit is shoveled into heaps in the summer, and in a comparatively short time another incrustation succeeds. The crude material carries from 5 to 20% of boric acid; and this crude borax is simply thrown into tanks of boiling water, into which chlorine or sulphuric acid is introduced. Here the alkali salts are dissolved and the boric acid liberated; after 24 hours the clear supernatant liquid is run off into recrystallizing tanks, and when cool white pearly scales are formed which are then washed with cold water. The Oregon works produce 400 tons of refined borax per year.

A 2,000,000-BARREL OIL RESERVOIR has been completed at Beaumont, Texas, by the Southern Pacific Railway Co.

A NATURAL SOAP MINE is noted by U. S. Consul-General John L. Bittinger, of Montreal, as having been found in the foothills near Ashcroft, British Columbia. This natural washing compound, containing borax and soda, is found on the bottom and sides of several soda lakes, but no two analyses of the compound agree as to composition, the same sample having been reported as containing 16 and 26% of borax. A syndicate has been formed to put the product on the market, and about 275 tons have already been taken out of one of the lakes, and this one lake alone is said to contain 20,000 tons of a substance equal to the washing powders in common use.

A RUSSIAN OIL WELL in the Baku Peninsula, says U. S. Consul James C. Chambers, of Batum, produced 180,000 barrels per day for the first two or three days, and continued to flow until it produced over 2,000,000 barrels. This well was 1,813 ft. and finished with a 14-in. pipe. While such a well would mean a fortune in a few days in Texas, Mr. Chambers says that the owner of this well that produced 2,000,000 barrels in 30 days lost money at it. But the well was on government property and subject to a tax of 5 kopeks per pood, or 2½ cts. on 36,112 lbs.; and further a high wind carried the oil spray over a neighboring village and the well owner had to pay for repainting about all the houses in the village. The big well was stopped by striking another in the near neighborhood at the same depth, which produced 1,000,000 barrels in December, and was still flowing at the rate of 25,000 barrels per day when Mr. Chambers wrote to the State Department.

COAL MINING IN CHINA is made the subject of late reports by several U. S. Consuls in that country. The coal mines of Tongshan and Lingsi are the only mines in China worked upon modern methods. The Chinese Engineering and Mining Co. does about two-thirds of this work by contract; and the contractors pay the miners from 9 to 12 cts. gold for eight hours' work, and add to this a small percentage of the net profits. The contract prices for mining amount to 12½ cts. gold per ton for dust coal, and 30 cts. for lump coal. Anthracite coal is mined on the Yalu River, is hauled 50 miles by mule carts, and sold to the natives at $1 gold per long ton. All Southern Manchuria seems to be remarkably well supplied with coal; but it is only locally worked and used. The Chinese Engineering and Mining Co., first mentioned, by putting a European staff at work has increased the monthly output from about 6,000 tons mined by natives, to 50,000 tons in December, 1901, and 54,000 tons in January; the engineers expect to raise this figure to 90,000 tons in May, and later to 150,000 tons per month. The company proposes to develop the ice-free port of Chinwangtao, and a branch railway now connects the mines with the Pekin-Shanhaikwan line.

THE MOUNT BEACON ASSOCIATION has completed a cable railway to the top of Mount Beacon, a mountain near Fishkill, N. Y. It gives access to lands proposed as a summer resort. The road is 2,150 ft. long, and the grade is said to be 64% for 800 ft. of this distance; and two 175-HP. electric motors at the top operate the drums on which the 1⅜-in. hauling cable is wound. The chief dependence for safety is upon powerful brakes on the cars, and upon strong steel jaws that would enter the wooden guard rails in case the cable broke.

THE LUXEMBURG BRIDGE FROM THE VIEW-POINT OF AN AMERICAN DESIGNER OF MASONRY ARCH BRIDGES.

By Albert W. Buel.*

In Engineering News of March 6, there was a very interesting discussion of the Luxemburg Bridge containing points exceedingly well taken, some of which may be strengthened by reaching the same conclusions from different view points. Some of the questions raised are as yet by no means determined and different opinions regarding them may be compared with profit, and one or two features of the design and construction seem to have escaped the consideration they should receive.

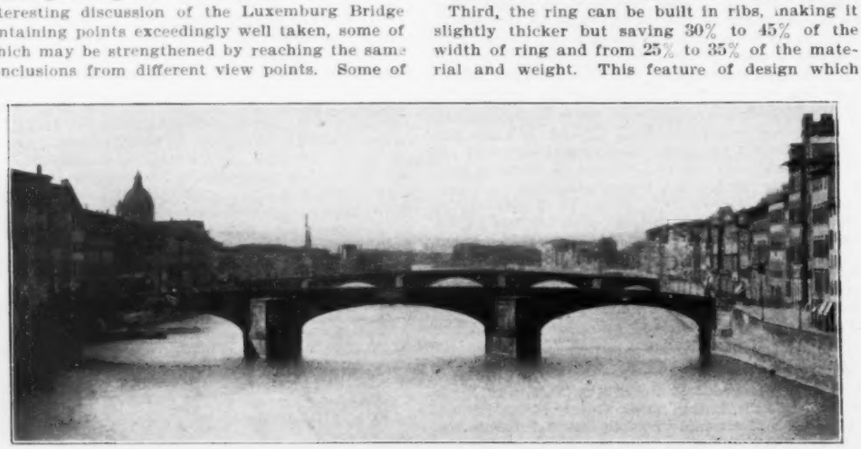

FIG. 1. GENERAL VIEW OF HOLY TRINITY BRIDGE OVER THE RIVER ARNO AT FLORENCE, ITALY.

To bury the springing lines of an arch is generally inexcusable, and the sketch (Fig. 2, Eng. News, March 6, p. 193) is a much better solution of the problem because it fits the ground and the natural conditions. The span, necessarily long, is made longer than any of the conditions require, and is, therefore, from this very point, a tour de force, and because of these conditions it so appears. The question remains, however, is a masonry arch span of 277 ft. a tour de force per se? This, I think, can be answered in the negative.

If we compare all the great masonry arch spans and the dates of their construction, with the relative facilities and quality of materials existing at the time they were built, it will appear that engineering has made little progress in this line. In fact, excepting the Luxemburg bridge, the bridge at Trezzo has held the record.

Some of the Roman arches, built in the early part of this era had spans as long as 140 ft. Then we record the Trezzo arch, 251 ft., built A. D. 1380; the Pont-y-Prydd, 140 ft., built A. D. 1755; the Cabin John Bridge, 220 ft., built A. D. 1860; the Grosvenor bridge, 200 ft., built A. D. 1833; the Jaremcze R. R. bridge, 213 ft., built A. D. 1892.

Excepting the Jaremcze bridge, all of these structures were built before we had the hard-burned high test Portland cement of the present time; before the elastic theory had been demonstrated; before the Austrian experiments had proven the elastic theory to be correct; and before the value of the combination of steel imbedded in masonry was understood or scientifically applied. These advances give us the following advantages in the construction of an arch of masonry.

First, with the Portland cements now on the market, we may either make the joints of our finely cut voussoirs nearly perfect, giving a ring practically homogeneous from skewback to skewback, or we may construct our ring of a concrete as good as the best stone that ever came out of a quarry, giving a monolithic ring with all its advantages.

Second, we may reinforce our monolithic concrete ring with steel bars or ribs placed so as to add the maximum of strength with the minimum of material and thus reduce the thickness and weight of the ring by such an amount that spans of a length much greater than could possibly be considered with cut stone voussoirs become undertakings of comparative ease. If any object to the appearance of a concrete ring—which is a matter of taste, cultivated or uncultivated—the ring may be faced with cut stone voussoirs, the concrete forming the sheeting just as very many cut-stone arches have been sheeted with brick without a word of objection having ever been made.

Third, the ring can be built in ribs, making it slightly thicker but saving 30% to 45% of the width of ring and from 25% to 35% of the material and weight. This feature of design which has been used in the Luxemburg bridge, and which alone is sufficient to provide for its excess of length over that of the Trezzo bridge of 1380, I believe was first suggested by Mr. Lindenthal in his discussion of a paper by Mr. Molitor. This same feature was adopted by Prof. W. H. Burr for his premiated design for the Memorial bridge at Washington, as well as for all his other designs for that bridge. In the latter, the total width of the five arch ribs is only 57% of the total width of the arch; while in the Luxemburg arch, the two ribs are about 67% of the total width, showing a much less saving than in Prof. Burr's designs.

Another feature of design which is common to both the Luxemburg bridge and the Memorial bridge design of Prof. Burr's, is the use of hollow spandrels. I cannot give the date of the first arch bridge with hollow spandrels, but this construction probably originated in France in the early part of the 18th century. The Pont-y-Prydd, built by Wm. Edwards, over the River Taff, in Wales, is an interesting example of this and the earliest in Great Britain (1755). Edwards built a bridge in 1751 which fell down when the centers were struck. The span, as given above, was 140 ft. He then replaced it, using hollow spandrels, [illegible] ducing the load over the haunches, and the [illegible] is standing to-day. The Jaremcze bridge [illegible] other example of the hollow spandrels. W[illegible] great reduction of weight on the ring du[illegible] use of hollow spandrels is considered, the L[illegible] burg bridge, with its parallel ribs and hi[illegible] cement mortar joints fails to impress me [illegible] excess of only 26 ft. in length over the [illegible] bridge, or with its 57 ft. excess in length [illegible] Cabin John bridge, or even with its 77 ft [illegible] length over the Chester bridge. Indeed, th[illegible] signers and builders of these older bridge[illegible] many more difficulties to meet, and deser[illegible] greater credit, for they were pioneers in a[illegible] explored field. They did not have anythin[illegible] ter than a merely approximate theory for th[illegible] lution of the stresses in an arch ring, while t[illegible] we solve these stresses with as great accuracy [illegible] certainty as we do those in a simple truss [illegible] girder. This is a matter of the very greatest [illegible] portance when dealing with long spans.

Exactly similar causes as those above [illegible] tioned have resulted in the increase of length [illegible] metal bridges until now we have spans of 1,[illegible] ft. and designs for spans of 3,000 ft. The strength of the material has not increased much more in the one case than in the other, comparing wrought iron with natural cements; high steel with high test Portland cements, and cable wire with concrete-steel combinations.

In taking up the economic side of the question, I will compare the estimates for Prof. Burr's designs for the Memorial bridge at Washington, having assisted in preparing them. These give the best data available for the purpose, having been worked out with great care and in considerable detail, and because designs Nos. 1 and 3 are similar in respect to width and loads and the length of the span of the arches (283 ft.). The figures given below, in each case, will include the main arch spans complete, the bascule and the piers and abutments, in all about 1,500 feet of bridge proper, and will exclude all approaches, pavements, railings, towers and ornamental features. With the exception of the bascule span, which is exactly the same in both cases, and amounts to only $215,690 of the total cost, practically all the items included are those that relate to the variable cost between four steel arch spans of 283 ft. and four masonry arch spans of 283 ft.

Design No. 1, steel arches	$1,600,000
" " 3, masonry arches	2,050,000
Excess of masonry over steel	450,000

This being only first cost is not, however, a fair comparison to the masonry design. Taking the

FIG. 2. NEAR VIEW OF ONE SPAN OF HOLY TRINITY BRIDGE, SHOWING THE PONTE VECCHIO IN THE DISTANCE.

*32 Broadway, New York city.

annual cost of maintenance and renewals of the steel structure at $4,000 (¼ of 1% of first cost), and capitalizing it at 3% we get $130,000; to add to a first cost of $1,600,000, giving a total cost of $1,730,000, while the total cost of the masonry structure with $10,000 added for capitalized maintenance is only $2,060,000, which exceeds that of the steel structure by only $330,000 or about 20%. It should be noted, however, that this excess is only about 10% of the total cost of the entire bridge with its approaches, pavements, etc.

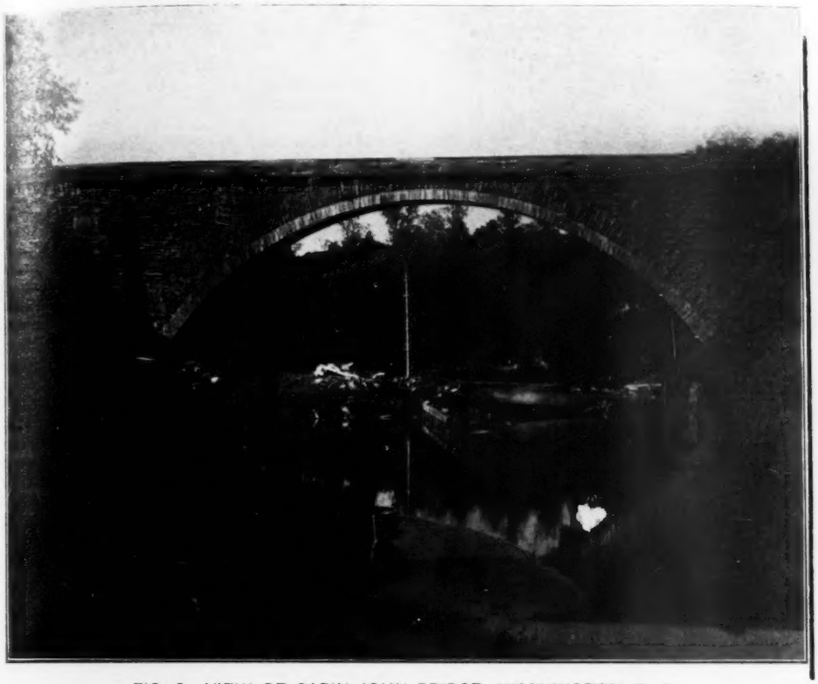

FIG. 3. VIEW OF CABIN JOHN BRIDGE, WASHINGTON, D. C.

No one can claim that this is an excessive percentage to pay for the aesthetic treatment of a public work, when it is exceeded in many commercial structures. But this is not expended for the aesthetic treatment alone. Durability is worth something, even after the cost of renewal has been capitalized against the steel structure. We know that masonry arches will last for 2,000 years and possibly 4,000, but it would be a rash prediction to say that a steel structure is good for much over 200 years.

Up to spans of 300 ft., there should be no question as to the propriety of using masonry arches when the circumstances warrant their consideration at all. With a sufficient height available the practical limit of masonry spans is considerably beyond 300 ft.

Before leaving this comparison it may be interesting to show that for design No. 2—the premiated design—the same length of bridge proper, exclusive of approaches, about 1,500 ft., was divided into seven instead of into five parts, as in designs 1 and 3 mentioned above. Six masonry arches of 192 ft. each, are placed three on each side of the bascule. Including and excluding the same items as above, we get a first cost of a little less than $1,000,000 or $600,000 less than for the four steel spans. It is fair to mention that this was for only a single deck, while designs 1 and 3 were double decked, so that in this case the loads are not similar. But it is near enough to show that six 200-ft. masonry spans do not cost as much as four 300-ft. steel spans, and that the great excess in cost of masonry over steel is largely imaginative. If it is so that the typical steel viaduct is a more logical structure than masonry, why are some of the best railways, guided by the most intense commercial spirit, adopting masonry wherever possible?

The suggestion of Engineering News, of a parabolic ring instead of what seems to be a three-centered curve in the illustrations of the Luxemburg bridge is a very good suggestion, because the right curve for a flat arch is yet an open question. It is certain that a segmental curve does not fit the lines of pressure for an arch much flatter than a semicircle. The apparent success of existing segmental arches may be partly accounted for by the abutments acting to some extent as a part of the arch ring, somewhat as if a part of the ring were hidden in a false extension of the abutments. The Luxemburg bridge is a good illustration of the transition stage of this defect and the criticism of this point by the Engineering News is very well taken.

One of the best and most experienced engineers in America in masonry arch design uses three center and five center curves. The designer of the Luxemburg bridge may have reached his result by a similar method. After many trials of various multi-centered curves I have found that a ring with an elliptical intrados and a segmental extrados fits the line of pressure as nearly as anything I can get for flat arches. The true line of the arch is the neutral axis of the ring. Just what kind of a curve this is, bisecting the ring between the ellipse and the segment, I have been unable to solve. It would be interesting and valuable information to have, and perhaps some of your mathematical correspondents can give it. The form of the ring is most important. If the line of pressure nearly corresponds to the neutral axis the moment tending to deform the ring is small and the thrust is easily taken care of. Failures and open joints are almost always due to excessive moments which are avoidable. An illustration rather interesting is found in the Memorial bridge designs referred to above. The 283-ft. spans of design No. 3 have elliptical intradoses and segmental extradoses. The moment and consequent flexure is almost nil. The 192-ft. spans of No. 2, the premiated design, have segmental intradoses and consequently high moments. It was actually much easier to provide for the stresses of the 283-ft. spans than for those of the 192-ft. spans, and the former are far better distributed. The 283-ft. elliptical rings are easier to build and have more favorable unit stresses than the 192-ft. segmental rings, which furnishes another reason why longer masonry spans are logical and practical, as all of the older bridges mentioned were segmental.

It has always seemed to me that if we can get a conic section curve to meet the requirements of the curve of pressure as found by the elastic theory, that we will be nearer to the true expression of the arch than by any arrangement of multi-centered curves. There are only two styles of classic arches, i. e., the Roman or semicircular and the Gothic or pointed. The Moorish and foliated styles are simply variations of these. They are both correct forms, mathematically and artistically, but both have limitations in application. They both require a considerable rise and the Gothic arch should only be used with a heavy load on the crown.

We require in many, if not the majority of cases, a correct form for a comparatively flat arch. The segmental arch is a chopped off Roman arch, and is no arch at all—it is only a piece of an arch. The semi-elliptical arch is very fairly represented by the Holy Trinity Bridge in Florence, built in 1569, and as it has continued to "give pleasure to the artistic mind" for nearly 350 years above most other bridges in the world, it may be taken as an artistic model for flat arches. The Roman arch is tangent to the vertical at the spring and the Gothic is so generally, and always nearly so. The elliptical is the only flat arch which agrees in this respect with its classic predecessors.

The impression that the Luxemburg bridge gives, as so well expressed by the Engineering News as, "Straining the resources of engineering knowledge to wring from stone masonry its utmost possibilities as a structural material," is, I think, wholly due to defects in its design, which have been pointed out. The Washington monument is a structure which well illustrates this very point. It is a shaft of masonry very near the limit of height for the stone to resist crushing in the lower courses. In fact, many of the stones in

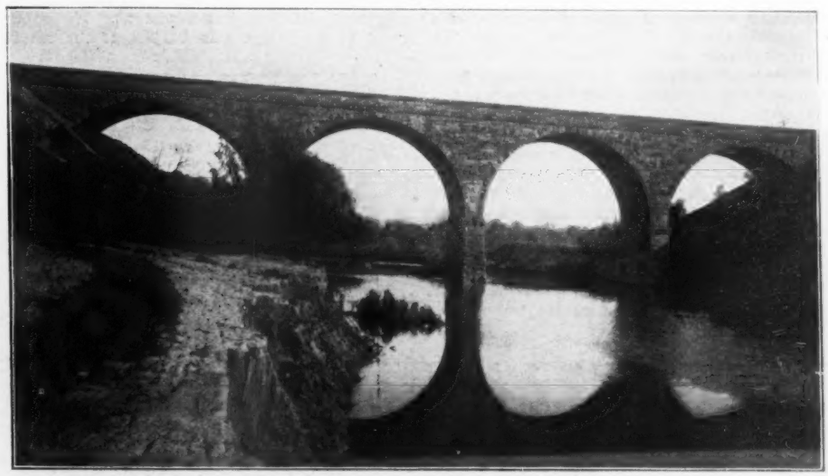

FIG. 4. PAINESVILLE VIADUCT, LAKE SHORE & MICHIGAN SOUTHERN R. R.

the lower courses are spalled on the edges by the pressure. Yet no one has criticised it as unpleasing, quite the contrary.

As a matter of fact, the Luxemburg bridge is not very near the limit, yet it appears to be. When all proportions are correct, when the entire design fits the local conditions and surroundings, the eye will scarcely notice the size, no similar structure being in view to compare with it. Size is not a very great element in pleasure to the artistic mind, but proportion and harmony are more important.

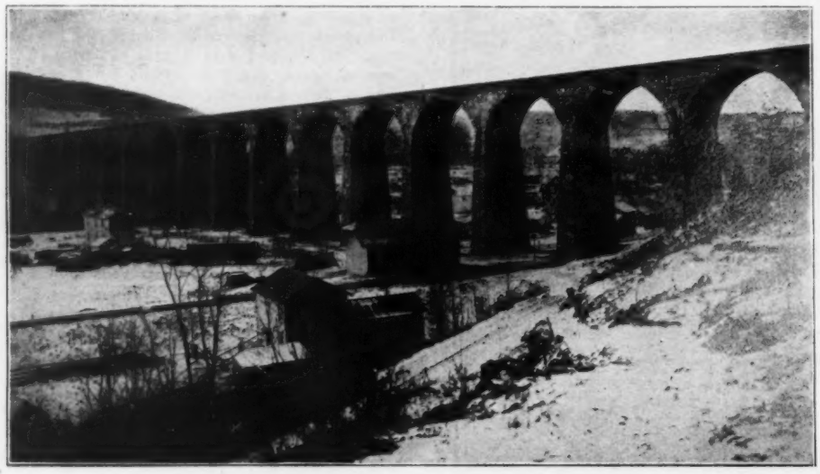

FIG. 5. STARUCCA VIADUCT, ERIE R. R.

Returning to the description of the construction of the Luxemburg bridge—the method of constructing the ring in many segments was a necessity on account of the center used. Never before has a long span masonry arch been built on such a flexible center. It is certainly interesting, but it seems rather risky. Ordinarily the center is the part of the work on which it pays to be the most liberal.

With the hundreds of concrete-steel bridges in France and Germany which have been attended with the greatest success—some of them very long spans, the use of cut stone voussoirs for this ring can only be explained by some such local, political or industrial condition as is explained by Mr. John N. Bellwald in Engineering News of March 27. Certainly concrete-steel would have effected a considerable saving, but would have required a more rigid center with more supports. Taking everything into consideration, we may learn from the Luxemburg bridge how not to do a number of things, but how to do very few, if any.

The accompanying illustrations show some of the structures to which reference is made above, and also give views of some rather notable railway viaducts which have been built of masonry The following are brief descriptions of the bridges illustrated:

THE PONTE DELLA SANTISSIMA TRINITA (HOLY TRINITY BRIDGE) over the river Arno at Florence, Italy, was built in 1569 by B. Ammanati. It is constructed of white marble. The center span is 95.75 ft. in the clear, with 15.12 ft. rise. The two outside spans are 95.25 ft. in the clear with 14.83 ft. rise. The clear water way is 270.5 ft. The piers are 26 ft. wide, measured on the center line of the bridge, and the bridge is 33.75 ft. wide, measured parallel to the axis of the arches. The depth of the key is 2.75 ft. This bridge is illustrated by two views, Fig. 1 showing a general view of the entire structure; and Fig. 2, looking under one of the arches with the Ponte Vecchio in the background, shows considerable detail of one span. Four graceful statues adorn the four corners at the ends of the bridge. Some criticism has been made of the width of the piers (26 ft.), but in general the bridge has been very much admired for nearly 350 years, and unless it had considerable merit this could not have been so.

THE PONTE VECCHIO OR JEWELERS' BRIDGE is said to have been built in 1345. It consists of three segmental spans, each 96 ft. long, with 19.2 ft. rise. It derives its chief interest from the arcade over the walks and the overhanging houses which are built along its sides and which are used mostly as goldsmiths' shops. As may be seen from the illustration, it crosses the Arno just above the Trinity bridge.

CABIN JOHN BRIDGE of the Washington (D. C.) Aqueduct, shown by Fig. 3, was built about 1860, by Gen. Meigs, U. S. Army. It has a clear span of 220 ft., and a rise of 57.25 ft., with a radius of 134.25 ft. The depth of key is 4.16 ft. The backing, for some distance above and beyond the ring proper, is laid up with radial joints, thus in effect making it a multiple ring arch. It has been the longest existing masonry arch in the world for over forty years, which distinction is now about to be captured by the Luxemburg bridge.

THE GROSVENOR BRIDGE over the Dee at Chester, England, was designed by Thomas Harrison. The contract was taken and signed by James Trubshaw on Jan. 25, 1827; and he completed the work in 1833. The engineer of construction was Jesse Hartley.

The bridge has a clear span of 200 ft. and is the longest in England. The rise is 42 ft. and the radius is 140 ft. The depth of the key is 4 ft. The material used was sandstone. The contract price of the bridge was £29,300, of the embankment £7,500, and of the total work £36,800. This bridge held the record as the longest existing span in the world for 27 years.

THE PAINESVILLE VIADUCT on the Lake Shore & Michigan Southern Ry., Fig. 4, consists of four semicircular arches each of 80 ft. span, and 3 ft. thickness of ring. The height of the roadway is 90 ft., and from the soffit of the arches at the crown to the top coping is 10 ft. [illegible] are 10 ft. wide at the springing line [illegible] one exception that an odd number of s[illegible] have been more pleasing, it is not easy [illegible] the design could have been improved. [illegible] debted to Mr. E. A. Handy, Chief Engi[illegible] L. S. & M. S. Ry. for the descriptive da[illegible] Mr. I. C. Horton, formerly chief draftsm[illegible] road, for the very excellent photograph

THE STARUCCA VIADUCT on the [illegible] Railroad, Fig. 5, was built in 1847. It is 1,2[illegible] long and 110 ft. high. The span of the flat [illegible] es is 51 ft. This bridge was built for a sin[illegible] track of the old Erie broad gage, but now a[illegible]mmo dates a double track road of standard ga[illegible] without any addition to its original width. Th[illegible]ches are apparently elliptical, but a better [illegible] ture would have been secured by the use of [illegible]ecir- cular arches, thereby reducing the heigh[illegible] of the piers and the horizontal thrust. In gener[illegible] when there is ample height available, as in this case, semicircular arches should be employed.

THE WISSAHICKON BRIDGE on the Philadelphia & Reading Railroad, Fig. 6, was built in 1881 and 1882, by Mr. C. W. Buchholz, now chief engineer of the Erie R. R., and to whom I am indebted for the photographs and description of this and the Starucca Viaduct. The Wissahickon bridge has five spans of 70 ft. each. The rise of the arches is 23 ft., width 28 ft., thickness of rings 3 ft., and the width of piers at springing 6.5 ft. With the four 10-ft. arches, two over each abutment, the total length of the bridge is given at 510 ft., its height at 80 ft. above the driveway, or 103 ft. above its foundations. It contains 15,400 cu. yds. of Talcose slate masonry and cost $375,000, or a little less than $3 per horizontal sq. ft.

A WATERPROOFING PROCESS FOR STONE, known as the Caffall paraffine process, is being applied to the exterior stone facing of the new 16-story "Tribune" Building in Chicago. The stone is heated by portable furnaces and a composition then applied which seals the pores or cells and thus prevents dirt from getting into the stone. Any dirt settling on the surface can be washed off. The first treatment tends to darken the stone, but a subsequent treatment brings back the natural color. The same process is said to prevent the white efflorescence which so frequently disfigures brick work. This is the process which was adopted some years ago for preventing the disintegration of the Egyptian obelisk in Central Park, New York city, the stone having commenced to crumble under the effect of climatic influences. Since then it has been used on a great number of office and public buildings, residences, etc., and it is claimed to be applicable for the waterproofing of tunnels and subways. The process is owned by Caffall Brothers, Chamber of Commerce Building, Chicago, and Caffall & Co., Merchants Building, New York.

FIG. 6. WISSAHICKON BRIDGE, PHILADELPHIA & READING R. R.

PRECIOUS STONES FROM U. S. SOURCES, to the value of $289,050, were produced in the year 1901. The largest item—$118,000—is credited to turquoise; sapphire—$90,000 comes next; tourmaline, $15,000; quartz crystal, $10,000, and beryl, $5,000 succeed in importance. The list issued by the U. S. Geological Survey covers 32 items, and includes the diamond, ruby, emerald, etc., but in each case the value of these produced in very small.

STEEL WHEELWAYS FOR WAGON ROADS.

By Ira O. Baker, M. Am. Soc. C. E.*

Some data in Engineering News of March 6, 1902, on the tractive resistance of steel wheelways brought the writer a couple of inquiries as to the practicability of this form of roadway. So much attention has been given to wheelways in some quarters as to justify a somewhat full consideration of the subject here.

Wheeled vehicles are drawn by horses, the wheels should move on the smoothest and hardest surface possible, while the horses require a surface rough enough to give them a secure foothold and soft enough to be easy on their feet. These two opposite requirements are united only in wheelways, i. e., in roads in which two parallel rails of suitable materials are provided to receive the wheels, while the space between the rails is filled with a different material, on which the horses travel.

The Egyptians seem to have first discovered the value of stone wheelways. In ancient times wheelways were of considerable importance, and in comparatively recent times they were used in London and Liverpool; but apparently they have been abandoned save in some cities in northern Italy. At about the time of the introduction of railroads, stone wheelways, or tramways as they were called then, were common in England; and for the information of those who are nowadays advocating burned-clay wheelways, the following description is given of a tramway built by Telford on a hill on his celebrated Holyhead road.

> The blocks were of granite, 12 ins. deep, 14 ins. wide, and not less than 4 ft. long. A foundation for them was prepared by making an excavation, 8 ft. wide and 25 ins. deep. On its level bottom was laid a telford pavement 8 ins. deep, the interstices being filled with gravel. Upon this pavement were laid 3 ins. of broken stones, none exceeding 1½ ins. in their longest dimensions. On them was a layer of 2 ins. of the best gravel, over which a heavy roller was passed. Upon this the stone blocks or "trams" were laid to a very accurate level. Upon each side of the blocks was placed a row of paving stones of granite, 6 ins. deep, 5 ins. wide, and 9 ins. long. The remaining space between the outside of the lines of paving stones was filled up with hard broken stone, and the whole was covered with a top dressing of 1 in. of good gravel.

This construction differs radically from that proposed by the present advocates of burned-clay wheelways, and it must be assumed under the circumstances that the great Telford don't knew how to build a tramway.

Stone tramways were abandoned in the streets of Liverpool a good many years ago, for the following reasons:

> When the impervious asphalt-jointed pavements were introduced, stone tramways were in some cases again employed; but it soon became obvious that they were no longer appreciated, as it was by no means uncommon to see the horse walking on the tramway stones drawing a load with wheels on the sets. The difference in traction was, in short, no longer perceptible. Stone tramways are the outcome of defective pavements. Under heavy traffic they are exceedingly difficult to maintain in a stable position. They induce exceptional traffic on the sets adjoining them, and therefore cause grooves to be formed. In crowded streets, even on bad pavements, their utility is but slight; while on good pavements they are worse than useless.

In recent years in this country, there have been many proposals to build steel wheelways. Numerous designs have been offered for the rail, most of which are highly impracticable, being difficult to manufacture, to lay, and to maintain. A number of sections would require the construction of expensive machinery for their manufacture before they can be tried even experimentally. Of course, this objection is not vital, but it will be well to carefully study past experience before going to any considerable expense to inaugurate a new experiment. Three or four sections, from 16 to 180 ft. long, have been built, and for a time were exploited in the newspapers; but even a little experience with them showed that they were practically valueless. A short section well cared for on dry ground under an experimental wagon gives little or no indication of the result under the ordinary conditions of actual service.

As showing the artificial character of the interest in steel wheelways, it should be noted that newspapers, official reports, and even Governors' messages during the past ten years, have contained glowing accounts of a 16-ft. section of such a road! Of course, the length of the road was never mentioned; nor was it ever stated that apparently the promoter ceased to defend his road after the first wet spell. Another significant fact is that the only public account there is of some of the more important official experimental sections, was made "two or three weeks" after the road was completed, and when it had been subjected only to a very light traffic during a dry time.

The only practical test of a steel wheelway ever made in this country was upon a section built in Chicago in 1901 from designs prepared in the Office of Road Inquiry of the U. S. Agricultural Department. The "rail" consisted of an 8-in. 11¼-lb. channel with legs down, having a 2½ × 2½ × ¼-in. angle, riveted to the outside leg by rivets 2 ft. apart and a 3 × 3 × ⅜-in. angle riveted to the inside leg. The top of the outside angle was flush with the top of the channel, but the inside one projected half an inch, to form a guide for the wheels. The 30-ft. sections of the rails were spliced by breaking the joints between the channels and the angles; and the rails were tied together transversely by bolting a short piece of T-iron under each rail and connecting the tees by a 1¼ × ⅜-in. bar. The design called for concrete between the legs of the channels; but as the constructing engineer (a competent railroad engineer) could discover no way of putting the concrete into place, a 2 × 8-in. pine timber was substituted. Each rail rested upon a mass of broken limestone 12 ins. deep and 12 ins. wide; and a 4-in. course of broken limestone was laid between and outside of the rails. About 2,100 ft. of double-track was constructed. The traffic over it was very great, consisting chiefly of wagons heavily laden with packing-house products.

Within two or three weeks after completion, the following defects had appeared: (1) The inner edge of the rail was considerably depressed, owing to the compression of the crushed stone under the rail. (2) The face of the rail was concave, except where the gage ties were attached, owing to the compression of the wood under the rail, and to the spreading apart of the lower edges of the rail. (3) The rails were bowed horizontally outward between the gage ties, owing to the wedging action of the crushed stone between the rails. (4) Pieces of broken stone were continually loosened by the horses' feet and kicked upon the face of the rail, where they were ground to powder. Under these conditions the tractive resistance was 40 lbs. per ton (see Engineering News, March 6, 1902, p. 184, Table 6, experiment No. 17); when the rail was scraped clean with a shovel, the resistance was 19 lbs. per ton (ibid., experiment No. 18).

It was then seen that the wheelway could not long endure under these conditions, and sawed railroad ties were inserted 3 ft. apart, and both flanges of the rails were spiked to the ties. After the above repairs, the rails maintained their lateral position reasonably well; but the rails deflected between the ties, and bad ruts soon formed adjacent to the rails, and the crushed stone was ground up to such an extent that the rail was almost hidden by the dust. At this stage the tractive resistance was 28 lbs. per ton. It is conceded that this steel wheelway has been anything but a success, and those responsible for it are only waiting for suitable weather to take it up.

ADVANTAGE OF STEEL WHEELWAYS.—The chief advantage which has been claimed by the advocates of the steel wheelway is a reduction of traction and a consequent increase of load. The advocates of this form of roadway frequently compare the traction per ton on earth roads with that on railways, and conclude that a horse can draw ten to twenty times as much on a steel wheelway as upon an earth road. This is erroneous for two reasons: (1) The steel wheelway advocated is not anything like as rigid and smooth as the ordinary railroad track, and can not be kept anything like as clean. Compare the three results above, particularly the last two. (2) The above claim as to the relative loads upon a steel wheelway and an earth road, neglects the effect of the footing upon the load that a horse can draw. Unless the space between the rails is paved, a steel wheelway would be but little better than an earth road in a muddy time—the time when the advantages of a steel wheelway are most desired. Further, unless the space between the rails is paved, it will be impossible for the vehicles to turn on or off the wheelway except at specified points. If turnouts are frequent, the expense will be great; and if they are infrequent, the inconvenience will be great. Therefore, it is safe to assume that any successful wheelway will have a permanently hard surface between the rails; and, consequently, the tractive power of a wheelway should be compared, not with an earth road, but with a road having the same surface as that between the rails of the wheelway.

Tests made by the writer on the steel wheelway above described show that the tractive resistance on such a road under the most favorable condition is more than that on a good brick or macadam surface; and that under ordinary conditions is about the same as that on a new plank road, or an ordinary brick pavement with concrete foundation, or a specially dressed granite-block pavement (see Table 6, p. 184, Engineering News, March 6, 1902). The resistance to traction on the steel wheelway is surprisingly large; and is doubtless due to the deflection of the rail as a whole and also of its surface immediately under the wheel, so that the wheel is continually compelled to climb a grade. In the light of these tests it must be concluded that unless the rail is very rigid and is kept clean, the reduction of the tractive power will not be very great. If the wheelway is made rigid, the cost of construction will be considerably increased; and if the rail is to be kept clean, the space between and adjacent to the rails must be covered with a practically indestructible surface which must be frequently swept.

In considering the relative tractive power required on different road surfaces, it should not be forgotten that the disadvantages of a grade increase as the tractive power increases. For example, if on a steel wheelway the resistance is 20 lbs. per ton, a 1% grade will double the resistance; while on a macadam road having a tractive resistance of 40 lbs. per ton, a 1% grade will increase the resistance only one-half; and if the traction on the macadam is 80 lbs. per ton, a 1% grade will increase the resistance only one-fourth. In other words, in the illustration above, on a steel wheelway, a horse can draw only half as much up a 1% grade as on a level, while on the first macadam road he can draw two-thirds and on the second eight-tenths as much up the 1% grade as on a level. Therefore the advantage of steel wheelways in decreasing the force of traction can be obtained only when the wheelway is level or nearly so.

Sometimes it is claimed that a steel wheelway makes a very durable road. As far as the steel rails are concerned, this is true; but it is not true of the surface adjoining the rails—whatever that surface may be. It is often assumed that the space adjoining the steel rails is to be paved with crushed stone. If this is the case, vehicles in turning out to pass each other will certainly wear ruts adjoining the rails, as is shown by the ruts worn in the most durable pavements by vehicles turning on and off of street-car rails. Further, it is well known that tracking is one of the most effective causes of the destruction of crushed stone roads—partly because the stones loosened by the horses' feet are not rolled into place by the wheels; and consequently the horses' feet will be more destructive on a macadamized steel wheelway than on an ordinary crushed-stone road. It has been estimated that the wear of ordinary macadamized roads due to the horses' feet was 1½ to 3 times as much as that due to the wheels. In consideration of the tendency to produce ruts adjacent to the rails, and of the fact that the wheels do not roll into place the stones loosened by the horses' feet, it is probable that the steel wheelways will at best add nothing to the durability of the adjoining macadam.

If the space between the rails is paved with a more durable material than macadam, the cost will be unreasonably great. The question of cost will be considered under the next heading.

DISADVANTAGES OF STEEL WHEELWAYS.—The most serious objection to a steel wheelway is that if it does not have a permanent footway for the horses, it is the least effective in a muddy time, i. e., when most needed; and if it does have

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†George F. Deacon, City Engineer of Liverpool, in "Proc. Inst. C. E.," Vol. 58, p. 28, session of April 29, 1879.

a permanently hard surface between the rails, the cost is unreasonable.

If a steel wheelway is laid in a crushed stone road, the additional cost will be (1) the cost of the metal, (2) the cost of placing the metal, including the increased cost of compacting the crushed stone, and (3) the increased cost of maintenance.

1. The steel wheelway described above requires about 121 tons of steel for a mile of single track. The cost delivered would probably not be less than $40 per ton, in which case the metal for a mile of single track will cost $4,840. In this connection it should not be forgotten that the experience with the above described wheelway shows that the rails are too light for any considerable traffic. 2. The cost of placing the metal will vary considerably with the design of the track and with the thoroughness with which the stone is compacted around the rails. Only experience can tell how much labor will be required to form an even bed for the rails, to bolt together the rails and attach the gage ties, and to consolidate the stone round the rails and ties; but it would probably amount to at least $500 per mile. There are 14 M. ft. of lumber in a mile of single track, which in place will cost at least $25 per M. or $350 per mile. If concrete is employed, it will cost two or three times as much as the lumber. A little allowance, say $50 per mile, should be made for the broken stone displaced by the wood and steel rail. The total cost of placing the metal will then be at least $800 per mile of single track. 3. It is impossible to accurately estimate the cost of maintenance. Only experience can determine the amount of labor required to prevent low joints and to fill up the horse paths and the ruts next to the rails. However, it seems reasonable to estimate that these expenses will amount at least to as much as the maintenance of an ordinary crushed stone road to carry the same traffic; and hence this expense may be neglected in this comparison.

The total cost of the wheelway proper would then be $4,840 + $800 = $5,640 per mile over and above the cost of a crushed stone road. What are the advantages to be derived from this expenditure? Only the possibility of a slight decrease in the tractive power. In considering the effect of a decrease in the tractive power, it should be remembered that such decrease is of value only to fully loaded teams; and further that such an advantage would accrue only to such fully loaded teams as haul their load from start to finish upon the steel wheelway or other equally good roads. The above cost would then be incurred for a very small advantage to a very small part of the traffic, and therefore there are very few circumstances under which the cost of a steel wheelway would be justifiable.

The amount of traffic is an important element as affecting the feasibility of a wheelway. If the traffic is small, the expense of the wheelway is unjustifiable, and if the traffic is great, wheelways cannot reasonably accommodate it. A single-track wheelway is undesirable, since vehicles turn out frequently in meeting each other, and since either the slowest team will regulate the speed of the procession or the faster team must turn out and go around. Turning out is difficult on account of the flange on the rail; and further, the sliding of a wheel against the flange causes the opposite wheel to tear up the macadam surface. A double-track wheelway would only partially remove the above objections; and ordinarily where there is traffic enough to justify a double-track wheelway, a universal wheelway, i. e., a first-class pavement, should be built. On the double-track steel wheelway referred to above, the writer estimated that more than half the vehicles turned out and went around the slow teams.

Another objection to any wheelway is that the smooth surface of the rail interferes with the footing of the horses. This in a considerable measure neutralizes the advantage of a smooth track for the wheels. Teamsters strongly urge this objection against the steel wheelway described above; and in addition claim that the guide flange injures the frog of the horses' feet. In connection with the last point, it may be interesting to note that the tramways built in London about 1825 were originally "built with guard stones projecting considerably on either side of the trams to form the wheelways. The guard stones were found to be objectionable, because the light traffic sought the projecting edge of the guard stone and wore a groove; and in relaying the tram [which was done three times in 21 years] the guard stones were omitted and the tramway made flush with the surface."*

Still another objection to a wheelway is that the gage of vehicles varies considerably; and consequently either the face of the rail must be very wide, which adds expense, or some vehicles cannot be accommodated. In almost any large city wagons are found with gages varying from 4 ft. 8 ins. to 6 ft. In this connection note that the tramways somewhat common in England 50 to 75 years ago, usually had trams 18 inches wide.

Finally, the conclusion is (1) that wheelways are out of date; (2) that they are more expensive and less effective than a good macadam road; and (3) that for a country road (the only position for which they are now advocated), where an ordinary crushed stone road does not suffice, a first-class pavement or a railroad should be built.

THE NORTH DIKE OF THE WACHUSETT RESERVOIR, METROPOLITAN WATER-WORKS.†

By Frederic P. Stearns,‡ M. Am. Soc. C. E.

Mr. Morison, in his interesting comparison of proposed designs for the Bohio Dam, has called attention to the resemblance between the design prepared by him and the north dike of the Wachusett Reservoir; and the writer, thinking that it might be of interest to the society to have at this time some further information with regard to this structure, which is now being built, has prepared the following description of its design and construction:

The preliminary designs of the dike were made by the speaker in 1894, in connection with the preliminary investigations for the Metropolitan Water Supply made by the Massachusetts State Board of Health. The design is an unusual one, and was the result of unusual conditions.

The Nashua River, at the site of the main dam, flows through a narrow gorge between rocky hills covered with a comparatively thin layer of drift material, and these hills rise far above the proposed water level of the reservoir. Between the hill on the north, however, and another hill nearly two miles to the west, there is a sandy plain having a general level about 15 ft. below the water level in the reservoir. In some portions of the plain small rocky hills or ridges rise above its surface; in other portions of the plain there are depressions and gullies.

North of Sandy Pond, which lies in a very deep depression, there is a deep cut through the plain, through which a small stream from this pond formerly flowed, but the building of a mill dam on a stream north of the plain backed the water up into Sandy Pond, and before the construction of the dike the water stood in the cut to a depth of from 5 to 12 ft. for a width of about 300 ft. The mill pond formed by the dam, including the arm connecting it with Sandy Pond, was known as Coachlace Pond. The bottom of this pond, at the point where the dike crosses it, is 65 ft. below the proposed water level. Upon making borings, it was found that the rock was far below the surface of the plain, in one place as much as 286 ft., so that it would be impracticable to reach it with any corewall.

The geological condition of the drift material over the rock, however, was much more satisfactory, as it was found quite generally that the material above the rock, with the exception of the upper layers, was a very fine and nearly impervious sand.

It had been decided, upon sanitary grounds, to strip the surface soil from the whole of the 6½ square miles comprised within the limits of the reservoir. The great depth of the reservoir and the conformation of the sides made it nearly as cheap to deposit this material at the site of the dike as elsewhere. The abundance of material available at a nominal cost, the fact that less water would percolate through a dike of great thickness than through a thin one, and the desire to provide absolute safety where such an enormous amount of water was to be stored, led to the adoption of a design which is characterized by very unusual dimensions.

DIMENSIONS OF THE DIKE.—The dike covers an area of 143 acres, and will contain 5,500,000 cu. yds. of material. It is two miles long on the water [illegible] full-reservoir level, 65 ft. high at the deepest [illegible] to this level, and has a maximum width of base of [illegible] ft.

In order to prevent the percolation of water through the upper layers of the plain, which in most [illegible] of coarse sand or fine gravel, an excavation [illegible] as the cut-off trench was made. This has a total length of 9,566 ft., a bottom width of 30 ft. and a maximum depth of 60 ft., and extends longitudinally under the crest of the dike. For 3,124 ft. the trench was excavated to the solid rock, and for the remaining 6,432 ft. [illegible] fine sand.

Wherever the borings indicated that the material at a level below the cut-off trench was somewhat [illegible], or where they showed streaks of the coarser [illegible] sheet-piling was driven in the bottom of the cut-off trench. This sheeting was driven for a total length of trench of 5,245 ft., leaving 1,187 ft. of trench without sheeting.

The portion of the dike easterly from a central rocky ridge, known as the easterly portion of the dike, presents the most interesting problems, and is shown in plan by Fig. 1 and in cross-section by Fig. 2. The characteristic features of the design are best shown on the cross-sections, one of them through the highest part of the dike and the others through the higher ground, where the water will have a depth against the dike of only 13 and 20 ft., respectively, but where the pervious character of the upper layers made it necessary to excavate a deep cut-off trench and fill it with impervious material.

The top of the dike is to be 17 ft. or more above the full-reservoir level, with a view to having it not less than 15 ft. above this level in the future after the material has settled.

The distance through the dike at the full-reservoir level is 189 ft. The down-stream slope, as a rule, is only 3%, but there are exceptions to this rule, and in some places the slope is as high as 6%.

The water face of the dam has a slope of 2 on 1, both above and below a berm 15 ft. in width and 13 ft. below the full-reservoir level.

MATERIALS USED IN THE DIKE.—As already indicated, soil stripped from the reservoir is used for by far the greater part of the embankment of the dike. This was found, by experiments which will be described subsequently, to be practically impervious, and except for a tendency to settle under pressure, to have a high degree of stability, and was therefore used for filling the cut-off trench as well as other parts of the embankment.

The water face of the embankment is to be protected above the berm with a layer of coarse gravel 7 ft. in thickness, resting on an embankment of either sand or gravel and covered with a well-laid paving 3 ft. in thickness of large stones embedded in and chinked with broken stone. The paving will extend from 8 ft. above the water level down to the berm, but will diminish in thickness toward its upper and lower ends. Below the berm light rip-rap may be used in exposed places, but as a rule a layer of coarse gravel will be sufficient.

It seemed advisable to give the cut-off trench a considerable width, and it was found to be both better and cheaper to excavate a trench with slopes rather than with vertical sides protected by sheeting. The open excavation, besides being cheaper to make, facilitated the work of driving sheet-piling. It furnished most of the material for the sand and gravel embankment at the water face of the dam, and its form is much more satisfactory than with vertical sides, where the trench is to be filled with a material which will compress as it is loaded more and more by the filling of material above it.

A secondary cut-off of somewhat smaller dimensions was provided for a part of the length of the dike, in the position shown on the cross-sections (Fig. 2), as an additional precaution, which could be provided at a small expense. A secondary embankment of sand and gravel was also added to meet a suggestion that the soil might lack stability.

The sheet-piling in the bottom of the cut-off trench was an important part of the work, which was accomplished in a very satisfactory manner. The borings showed the presence of fine sands, not wholly impervious, to a depth of 50 ft., in many places, below the bottom of the trench, which made it desirable to use unusually long sheeting if a method of driving could be devised which could give satisfactory results. The entire absence of stones or other coarse material indicated that good results might be obtained with a water-jet.

As it was impracticable to obtain single planks for sheeting of such great length, it was decided where the lengths exceeded 30 ft. to make the sheeting 6 ins. in thickness and to build it of 2-in. planks in substantially the same manner as the Wakefield triple-lap sheet-piling. Where the length was less than 30 ft., 4-in. grooved spruce sheeting was used. The 2-in. planks were of standard widths, planed to an even thickness and nailed together, except at the lower end, where they were bolted to resist water pressure. The piles were beveled at the bottom for a part of their width, so that they would hug the pile against which they were driven.

Spruce lumber, costing about $18 per 1,000 ft., B. M.,

*"Proceedings Institute of Civil Engineers," Vol. 58, pp. 52-53.

†Reprinted, with the omission of a number of illustrations, mostly half-tone, and references thereto, from the discussion on the Bohio Dam, "Proceedings American Society of Civil Engineers," April, 1902. The original paper, by Mr. Geo. E. Morison, M. Am. Soc. C. E., was printed in the "Proceedings" for January, 1902, and is the first instalment of the discussion in the "Proceedings" for March, 1902. An abstract of Mr. Morison's article, and of a portion of the discussion thereon, was given in our issue of March 13, 1902.

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was used for all except a short length at the bottom of the pile, where oak was substituted.

The piles were made of varying widths, generally between 10 ins. and 2 ft., and they were quite commonly from 40 to 50 ft. in length. The longest pile used was 67 ft., as they were left with their tops 5 ft. above the bottom of the trench in order to make a suitable connection with the soil filling, the greatest depth that a pile was driven into the ground was 62 ft.

For driving the long piles a pile-driver 50 ft. high under the leaders was used, and a water-jet was used in all cases. A large steam pump was installed at the bottom of the trench and a 6-in. wrought-iron pipe extended beside the pile-driver. From this the water was conveyed to the pile by a 3¼ or 2½-in. hose, or by a combination of the two, fastened by a detachable coupling to an opening which extended from the side of the pile, near its bottom, to a nozzle at the bottom of the pile. Various kinds of nozzles were used, to meet different circumstances. All of them furnished a heavy stream pointing directly down, and in general there was a stream spurting back toward the pile already driven, to wash away all sand from between the two piles. The large quantity of water used came to the surface, around the pile and also around the hose, keeping them free from pressure. The hammer of the pile-driver was used merely to overcome the friction of the pile against the wedges and the pile previously driven, and to overcome its buoyancy after the pile was driven to a considerable depth into the ground. When the pile had been driven to the full depth, a key fastened to a small wire rope was pulled, releasing the hose, which could then be drawn to the surface.

It had been thought before beginning the work that it might be desirable to drive two rows of sheeting, but the character of the work done was so satisfactory that it was concluded that one row would be sufficient. The largest number of piles driven at one place in any one day was 28. These piles were 20¼ ins. wide and between 45 and 46 ft. in length. They were driven into the ground from 40 to 41 ft.

The method of constructing and driving the sheet-piling was devised by and the work executed under the direction of Hiram A. Miller, M. Am. Soc. C. E., Engineer of the Reservoir Department.

Where the trench was excavated to the rock, the surface of the rock was carefully washed off with a hose, using water under a considerable pressure. All seams were carefully cleaned out, and, if small, filled with Portland cement mortar, or, if large, with bricks laid in the mortar. Brick cut-off walls were built where needed, and the entire rock surface was covered with two coats of grout, put on with whitewash brushes. It was the purpose in all this treatment of the rock to prevent any possibility of contact between a concentrated stream, which might possibly follow a seam or narrow channel in the rock, and the soil resting on the rock. The area of the rock surface treated in this way was 77,250 sq. ft.

At the extreme down-stream toe of the dam an embankment of very coarse gravel was formed. Its maximum section is 19 ft. high and 115 ft. wide at the base. This embankment is for the purpose of permitting any water which may find its way through the dike to escape without the sloughing of the material. If the quantity filtering should prove to be large, there is a coarse gravel stratum below the surface which could be tapped by means of driven wells to furnish an escape for water filtering through that stratum, but it is not anticipated that these wells will be needed.

COST OF THE DIKE.—The amount expended on the dike up to the present time is about $500,000, and about two-thirds of the work has been completed.

In looking at the cross-sections, one can hardly realize the relative cost of the different materials. The quantities and costs of the completed dike will be approximately as follows:

Soil	5,250,000	cu. yds., at $0.05	$262,500
Cut-off trench	542,000	" " " 0.20	108,400
Borrowed earth & grav.	200,000	" " " 0.20	40,000
Slope paving	50,000	" " " 2.20	110,000
Sheet-piling, pumping, consolidating soil with water, and other day's labor work			117,000
			$637,900
Engineering and preliminary investigations			120,000
			$757,900

The cost of the soil filling cannot, of course, be very definitely determined, as it is an estimated part of the cost of stripping and disposing of the soil from the reservoir, of which the total cost is from 20 cts. to 39 cts. per cu. yd. Of the total cost of construction, exclusive of engineering and preliminary investigations, amounting to $637,900, only about two-fifths is chargeable to the great mass of soil of which the dike is composed, and only one-sixth to the cut-off trench, which is of such unusually large dimensions. On the other hand, the slope paving and sheet-piling, which occupy very little space upon the cross-sections, together represent nearly one-third of the total cost. The cost per linear foot of dike, including the engineering and preliminary investigations, will be about $75.

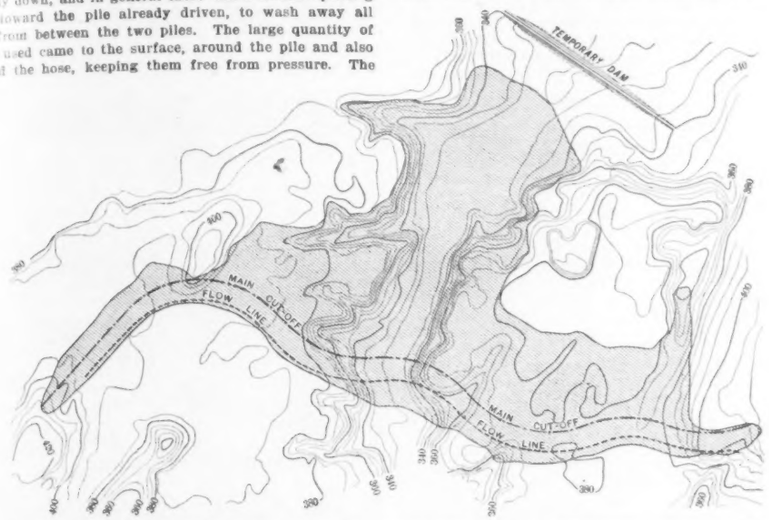

FIG. 1. PLAN OF NORTH DIKE OF THE WACHUSETT RESERVOIR, METROPOLITAN WATER-WORKS, AT CLINTON, MASS.

The Bohio Dam, as designed by Mr. Morison, has two-thirds the bulk of the North Dike, and his estimate of cost, exclusive of protective works, engineering and contingencies, is one-half greater than the cost of the North Dike.

INVESTIGATIONS ON WHICH THE DESIGN WAS BASED.—In making the original design of the dike, the writer was much indebted for information with regard to the filtration of water through fine materials to the investigations of Allen Hazen, M. Am. Soc. C. E., upon the physical properties of sands and gravels.* As indicated by Mr. Morison, Mr. Hazen determined the laws of the flow of water through sands and gravels with such accuracy that he was able to make a formula to express the results. By the application of this formula it was evident that the very fine sands found at a considerable depth below the surface would not permit enough water to pass through them if a dike of great width were constructed to cause a serious loss of water, and it was also found that the soil, which contains not only the fine particles of organic matter, but also a very considerable amount of finely comminuted particles, which the geologist has termed "rock flour," would be sufficiently impermeable to be used as a substitute for a clay puddle. The final design, however, was not adopted until after a most extensive series of borings had been made to determine the nature of the material at all depths in all parts of the plain, and an independent series of investigations had also been made with the materials to be found at and near the site of the dike to determine whether they would be suitable for use.

BORINGS.—The plain at the site of the dike was so wide that it permitted a large range in the choice of locations, so far as could be judged from surface indications, and it was necessary, therefore, to locate the dike by means of borings.

Owing to the great length and breadth of the territory to be explored and the great depth to the rock, this investigation was a most tedious and expensive one.

Borings were begun with three gangs on April 1, 1896; three more gangs were added on Aug. 15 of the same year, and the work was then continued without interruption, nearly all of the time with six gangs, until Dec. 13, 1897. A few additional borings were made in 1898. In all, 1,131 borings were made, having an aggregate depth of 93,553 ft., equal to about 17¾ miles. The average depth was 83 ft., and the maximum depth 286 ft.

The borings were the ordinary wash-drill borings, made by driving a 2½-in. pipe into the ground and washing the material encountered to the top of the pipe by a stream of water forced into the bottom of this pipe through a 1-in. wash pipe armed with a drill at its lower end.

In many instances the outer pipe was driven only through the layers of coarser material, and the 1-in. pipe was forced down into the fine sand, in which a hole could be maintained without a casing. Special methods were used to obtain trustworthy samples of the materials encountered.

Prof. W. O. Crosby, Geologist of the Massachusetts Institute of Technology, coöperated with the engineers in regard to the manner of conducting the borings, collecting the samples and interpreting the results.

The samples were classified largely by their appearance, but other features were also considered. When the water coming up the pipe while making the borings was discolored by clay, the fact was noted in the records, and in such cases a sample would be classified as more nearly impervious than if this note had not been made. The presence of clay in the samples could also be determined by chemical tests, even though a large part of the clay had been washed away and lost. Many of the samples were used for filtration tests upon a small scale, and these and other tests affected the classification.

Most of the borings at the easterly half of the dike were made where the ground-water level was from 35 to 50 ft. below the surface, and it is an indication of the comparative impermeability of the material that the water discharged by the 1-in. pipe generally came to the surface in a strong stream, notwithstanding there was a head of from 35 to 50 ft. tending to force the water into the ground, and that in many cases the sides of the very deep holes were not cased and the water could percolate into them as freely as the material would permit. There were places, however, where the stream of water flowing through the 1-in. pipe was wholly absorbed by the ground, and such places were noted in the records and served as an indication of the degree of permeability of the strata in which the water filtered away.

EXPERIMENTAL DIKE.—For the purpose of making experiments with a dike built of soil, and for other experiments, a building 70 ft. long and 25 ft. wide was constructed. Inside this building a water-tight wooden tank, 60 ft. long, 6 ft. wide and 8 ft. high, was built of 2-in. tongued and grooved pine planks, planed on both sides. Within this tank a dike of soil was constructed as shown in Fig. 3, 8 ft. high, 7 ft. wide on top and with a slope of 2 horizontal to 1 vertical on the up-stream side and 4 horizontal to 1 vertical on the down-stream side, except at the extreme lower end of the slope on the down-stream side, where there was a box 18 ins. high filled with gravel and having perforated sides, so as to allow the water to filter out at the toe of the slope without carrying the soil with it. Immediately over the top of the dike, for a width of 7 ft., there was placed 3 ft. of soil, so as to slightly consolidate the top of the dike and permit the water to be filled up level with it without overflowing.

After the dike had been formed, the upper end of the tank was filled with water to a depth of nearly 8 ft. In order to determine the pressure in different parts of the dike, horizontal pipes were laid across the tank through the soil, near its bottom, at intervals of from 6 to 12 ft. These horizontal pipes were perforated and covered with wire gauze, and at the end of each there was a vertical glass tube. As soon as the soil surrounding the pipes became saturated and under pressure, the amount of pressure would be shown by the height of the water in the glass tubes.

Experiments were made on dikes with the soil deposited in different ways: (1) When shoveled loosely into the tank without consolidation of any kind; (2) When deposited by shoveling it into water.

The soil, which had been thrown in loosely, settled as it became saturated by the water and became quite compact. After the pressure had been on for several weeks, the filtration through the dike amounted to only about 1 gallon in 22 minutes. It is interesting to note that it was five days after the water pressure was put on the dike before any water appeared in the sixth gage pipe near the lower end of the dike.

In the second experiment the filtration amounted to 1 gallon in 34 minutes.

In a third experiment, made under nearly the same conditions as the second, the rate of filtration was 1 gallon in 35 minutes.

In addition to the filtration test, the experiments showed that a light section of dike, composed of soil not rammed, would withstand a head of 8 ft. of water without failure.

FILTRATION EXPERIMENTS.—The experiments above cited show, incidentally, that extremely little water will filter through soil even when it is not compacted, but s

*Report of Massachuetts State Board of Health for 1892, page 541.

better test, particularly of the relative filtering capacity of soils and sands, can be obtained by filtering through cylinders of known area.

For this purpose five galvanized-iron cylinders, having tight bottoms, were provided, each 2 ft. 4½ ins. in diameter and 5 ft. high. The horizontal area, consequently, was 0.0001 acre. In the bottom of each cylinder there was a small wooden drain in the form of a cross, leading to a faucet on one side of the cylinder. Over the drain were placed stones about 2 ins. in diameter and on them smaller stones, diminishing in size, were added, until at the top of a 5-in. layer the material was either a fine gravel or coarse sand. Upon this layer, which would remove water freely, was placed the filtering material to be tested, to a depth of 3 ft. 8 ins. Within this filtering material, 4 ins. above its bottom and 4 ins. below its surface, were placed horizontal perforated brass pipes covered with wire gauze and extending through the sides of the cylinder. These were connected with ⅛-in. vertical glass tubes. The water to be filtered was applied in a steady stream at the top of the filtering material, so that it stood about 10 ins. deep, and was maintained at a constant level by admitting more than would filter, allowing the surplus to pass over an overflow. When the experiment was in progress the water appeared in the glass tubes and rose in each of them to a height due to the pressure of the water in the filtering material in immediate contact with the corresponding perforated brass pipe. The difference in the height of water in the glass tubes showed the amount of head lost by filtering the amount of water passing through the material. When it was desired to filter with a smaller loss of head than that due to a free outlet at the bottom of the tank, a stop-cock was partly closed, so that the water in the glass tube connected with the lower brass pipe would rise.

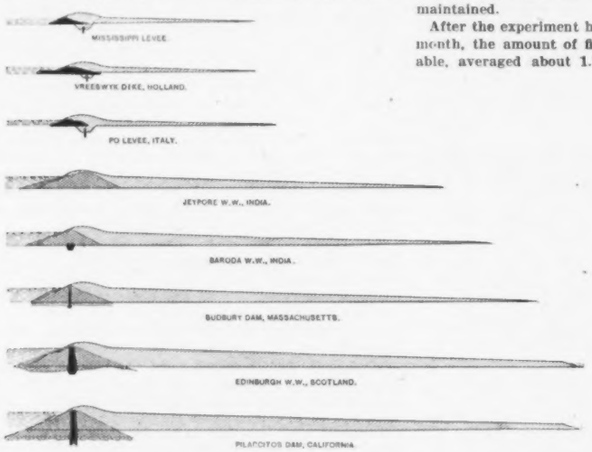

FIG. 4. NORTH DIKE OF THE WACHUSETT RESERVOIR COMPARED WITH EXISTING EARTH DAMS.

Experiments were made with gravel, sand of all grades, and soil from various locations, such as upland and intervale.

The experiments confirmed those of Mr. Hazen, by showing that the loss of head was directly proportional to the quantity of water filtered, and substantially confirmed his conclusion that the quantity filtered will vary as the square of the diameter of the smaller grains of the filtering material. The smaller grains referred to are those which separate the finer 10% from the coarser 90%, and are the grains which, according to Mr. Hazen's experiments, determine the "effective size" of the material for filtration.

To give all the experiments in detail, with the results of the mechanical analyses of the materials used, would require more space than it seems desirable to use in this discussion. The experiments with coarse and fine gravel are, therefore, omitted, and those with sand are grouped together in four grades in the accompanying table:

Amounts of Filtration, in Gallons per Day, Through an Area of 10,000 sq. ft. of Different Materials, with a Loss of Head of 1 ft. in 10.

Coarse sand, average of three experiments.....	2,200,000
Medium sand, average of six experiments.....	400,000
Fine sand, average of two experiments.....	90,000
Very fine sand, average of two experiments....	7,200
Soil, average of ten experiments.....	510

In order to express the results with a convenient unit of area, 10,000 sq. ft. has been adopted for the filtering area, and a loss of head or slope of 1 ft. in 10.

It is of interest to note the very great difference between the amount of filtration through the coarse sand and the very fine sand, and also to note how much less water filters through the soil than through the finest sand. The ratio of filtration through the various materials, as compared with the filtration through soil, is more clearly shown if each of the figures is divided by 510, so that the amount of filtration through soil will be the unit, as follows: Coarse sand, 4,353; medium sand, 784; fine sand, 176; very fine sand, 14; soil, 1.

There was some question, in view of the very small amount of water filtering through the soil, whether there might not have been some accumulation of air in its interstices, which diminished the amount of filtration, and it was therefore decided to experiment on the horizontal filtration of water through soil. For this purpose the large wooden tank used for the experimental dike, for a length of about 50 ft., was filled to the top with soil taken from an upland pasture. In order to retain the 50-ft. section of soil in place, and to allow the ingress and egress of the water without disturbing the soil, screens were placed about 55 ft. apart, and about 2½ ft. of porous sand and gravel were filled in between the screens and the ends of the section of soil. The soil was rammed in 3-in. layers, and special care was taken to prevent any water from following along the sides of the tank. At each end of the tank a space was left unfilled with earth, and one of these spaces was filled with water to a depth of 7.85 ft., while at the other end a free outlet was maintained.

After the experiment had been continued for more than a month, the amount of filtration, although somewhat variable, averaged about 1.7 gallons in 24 hours, indicating that the rate of filtration was as low as that obtained by the experiments in the galvanized-iron tanks. The tightness of this soil dam could be better appreciated by seeing it than by a statement of the figures. Although the section of dam was 6 ft. long and very nearly 8 ft. high, the leakage from the outlet amounted to only 32 drops in a minute.

To test the filtration through soil under greater pressure, a 10-in. pipe, to which a cover could be bolted, was used in the place of the galvanized-iron tanks, and the pressure upon soil 8 ft. in thickness was raised as high as 65 lbs. per square inch, equal to a head of 150 ft., and the results were not materially different from those deduced from the experiments in the galvanized-iron tanks.

As the soil to be used in the dike contained from 4 to 8% by weight of organic matter, the question arose as to whether, if this organic matter were to disappear, the soil would not become permeable. To determine this point comparative filtration experiments were made in the 10-in. pipe with soil as it came from the ground, and again after the organic matter had been burned out of it at the mining laboratory of the Massachusetts Institute of Technology.

Experiments were made for eight weeks with the natural soil and for eleven weeks with incinerated soil, with the result that about one-fifth more water filtered through the latter than through the former.

STABILITY OF SOIL.—The question having arisen as to whether soil when saturated would not be so mobile that it would not withstand the pressure of water, two experiments of the same kind were made; one with upland and the other with intervale soil. Each gave substantially the same results.

To make the experiments, the soil was filled into the large tank already described and compacted so as to form an embankment 2 ft. wide on top, 6 ft. high and with slopes of 2 on 1 on both sides. The embankment was then saturated by filling the tank on both sides of the embankment with water to a height of 5 ft. above the bottom. The top of the embankment was covered with 4-in. planks each 2 ft. long, and running lengthwise of the embankment. On these planks was placed a 12 × 14-in. hard pine beam, 6 ft. long, extending across the tank. Two large jack-screws were placed between this beam and another securely fastened above it, and were operated so as to press the planks into the soil. The apparatus did not permit of an exact measurement of the pressure, but it was between 28 and 42 tons applied to 12 sq. ft., making the pressure probably about 3 tons per sq. ft. The 4-in. planks were pressed down into the soil a little more than 6 ins., but the embankment was so stable that there was only a very slight bulging at the sides a short distance below the water level, showing that there was enough friction between the particles of which the soil was composed to make it stable.

This experiment, in addition to showing the stability of the soil, was also interesting as showing the amount to which soil would compress under heavy pressure. Immediately below the planks, where the pressure was one ton per square foot was greatest, the soil become hard and resisting.

The engineer in charge of the experiments found that his weight would easily push a sharp-pointed iron rod, ¾-in. in diameter, down to the bottom of the tank through the part of the embankment which had not been subjected to pressure, and that directly under the planks the soil was so compact that his weight would force it down only from 6 to 8 ins. This result is in line with other experiments subsequently made upon the compressibility of soil, and with practical experience where large banks of soil which have stood for a considerable time have to be excavated. The compressed soil needs a pickaxe to loosen it, and is very different from loose soil such as is dug from the ground or dumped from a train.

WILL FILTRATION CAUSE A FINE MATERIAL TO PENETRATE A COARSE ONE?—The question having been raised as to whether soil filled into a sand and gravel

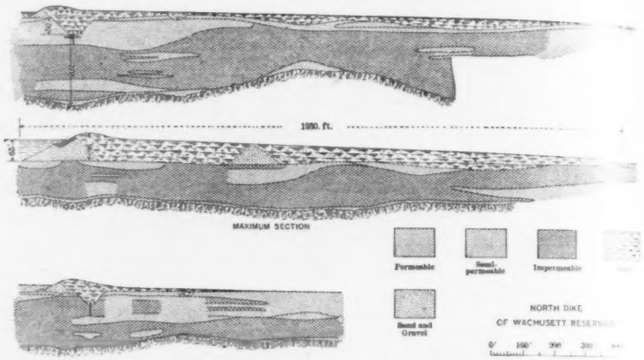

FIG. 2. CROSS-SECTIONS OF NORTH DIKE.

trench might, when the pressure was applied, penetrate the interstices of the sand, it was decided to make the following experiment, notwithstanding the fact that it is well known, from practical experience with water filtration, that fine particles are not carried to any considerable extent into the interstices of the sand. The 10-in. cylinder, already referred to, was filled to within 3 ins. of the top with medium sand and then 3 ins. of soil were put on the top, the end of the cylinder was closed, and water was applied, with a pressure of 65 lbs. per sq. in. The water was allowed to filter through the soil and sand for a week, and upon examination at the end of this time there was no indication of any movement of the soil or that any of it had worked into the sand.

PERMANENCE OF SOIL.—An engineer naturally dislikes to use, in an important and permanent structure, a material which is not permanent, and the question was, therefore, raised as to whether soil which is known to contain a considerable amount of organic matter would be permanent. It was thought best to refer this matter to experts, and Mr. Allen Hazen and Prof. W. O. Crosby were requested to report upon it.

They agreed in their separate reports that organic matter would disappear very slowly, indeed, except by oxidation with free oxygen present; that in the portion of the dike which was saturated there would be very little water filtering from the reservoir, and the free oxygen obtained from this source would, therefore, be very small; also, with regard to oxygen contained in the rainwater which fell upon the dike, that it would be removed from the water as it filtered through the upper layers of the soil. Their main conclusion, therefore, was that the process of oxidation below the line of saturation would be extremely slow, requiring many thousands of years for the complete removal of the organic matter, and that the tightness of the dike would not be materially affected by any changes which are likely to occur. There will, no doubt, be some settling of the material of the dike in the course of years, mostly in the portions above the line of saturation, but, as due allowance has been made in the height of the filling for this settlement, it will do no serious harm. Below the line of saturation, which is the only place where changes in the condition of the soil can do any harm, there is a heavy pressure upon the soil, caused by the weight of the upper layers, and the material is sure to remain compact, even if some of the organic matter should be removed by chemical action. Mr. Hazen, in his report, suggested that this heavy load was likely to push the grains of mineral matter nearer together and to crowd the organic matter into the interstices, so that the disappearance of organic matter from the compressed soil

cause less settlement than from soil that was not so saturated.

The bottom of the deeper portions of the cut-off and of the maximum section of the dike there is a pressure of about 4 tons per sq. ft. upon the bottom soil, and of about a ton per sq. ft. upon the soil at the limit of saturation. Attention has already been called to the experiments which showed that the soil with all organic matter removed was practically water tight.

COMPARISON OF DIKE WITH EXISTING DAMS.—Fig. 8 shows eight comparisons between the North Dike and existing earth dams. Where the existing dam had less height than the maximum section of the North Dike, the comparison is made with a section of the dike having the same height up to the water level; where the existing dam was higher than the North Dike, the comparison is made with the maximum section of the dike. The black, heavily-shaded portion is in every case the existing dam, while the light shading represents the North Dike. The black central walls shown in connection with several of the existing dams are clay walls, in every case excepting the Sudbury Dam, where the wall is of concrete, plastered on its up-stream face. Similar comparisons were made with many other dams, and they showed similar results, namely, that the dimensions of the North Dike are far in excess of those of any structure of this kind which has been built. The Jeypore Dam has no core-wall, and is composed largely of sand and other light earth, resting upon a foundation of the same general character.

STABILITY OF DAMS.—At this time, when not only the great dams of the Isthmian Canal are under consideration, but there is much discussion in the engineering papers with regard to the stability of earth dams, and many accounts of the failure of masonry dams, it seems opportune to discuss briefly the stability of dams in general.

In order that a dam may fail it is obviously necessary that the material of which it is composed must move, either as a whole or in part, under the pressure or the flow of water. If a dam has great bulk and weight and a reasonable degree of stability it cannot fail by sliding on its base or by overturning as a whole, but must fail by some separation of the parts.

In the case of earth dams, failures have generally occurred from the wearing away of the material by a current of water, either flowing over the top of the embankment or through some passageway formed in the dam, in connection with a pipe through it or by some burrowing animal.

Occasionally, an earth dam with steep slopes, or one with flatter slopes in which the material on the down-stream side retains the water, will fail through the sloughing of the down-stream slope, but the speaker does not know of any instance in which the earth confined in or under a dam has been moved by the filtration of water through it.

The tendency among many engineers at the present time seems to be to pay more attention to the materials and method of construction than to bulk. Many masonry dams have been built upon a somewhat theoretical cross-section, without taking into account the upward pressure of water on the base of the dam and the many uncertainties as to what will actually occur when the dam is put under great pressure; as a result, the failures of masonry dams over which large quantities of water flow have been numerous. A comparison of the sections of such structures with the sections of the upper part of masonry dams which rise to a considerable height above the water, like the New Croton Dam and the Wachusett Dam, shows the relatively small weight of these spillway sections, and no amount of good workmanship will compensate for the failure to provide sufficient weight.

The writer has frequently been asked why he did not use Portland cement throughout the whole section of the Wachusett Dam, and his answer has been that, except in the parts of the dam where Portland cement has been used, natural cement will withstand all the pressures that can occur, and that with natural cement the same amount of money will build a dam of so much greater weight as to provide absolute security against sliding or overturning.

In the case of earth dams, the bulk is necessarily large, so that with well-constructed dams there are practically no cases of failure, except where the material has been washed away by a current of water.

In the case of the North Dike, the top being 15 ft. or more above the full-reservoir level and 10 or 12 ft. above any height which is likely to be reached during freshets, it is impossible that any water should ever run over the top. There are no pipes or structures through the dike to form channels through which any current of water could flow to remove the material. The dike is protected against burrowing animals by the paving and broken-stone protection on the water face, extending 8 ft. above and 13 ft. below the reservoir level, but even without this protection an animal would have to burrow 289 ft. to reach a point on the down-stream side of the dike 3 ft. below the water level.

There is another advantage in having a great mass of material above the water line of the reservoir, which is worthy of consideration, viz., that in the event of the brief control by an enemy or a mob it would be impossible to remove enough material to cause the failure of the dam.

The filtration through the dike will be so slow, and will take place where there is such a load upon the material, that there can be no possible movement of the particles on account of this filtration.

The precautions taken at the down-stream toe of the dike are such that the water cannot accumulate and cause it to slough, and even if any trouble of this kind should occur it would take place so far from the principal part of the dike that it could be remedied before any trouble could possibly occur.

In the case of the plan proposed by Mr. Morison for the Bohio Dam, the writer believes that it cannot fail to be safe if the crest is built so high that floods or waves cannot possibly overtop it, and if the dam is properly built. Such a large mass of material, in a dam which is constructed so as to be less pervious near the reservoir than toward the down-stream toe, cannot be pushed or washed away. If a large amount of water should filter through the material under the dam and come out at the lower end, it may cause some movement of material in the vicinity of the toe unless precautions are taken to prevent it. Few engineers who deal with filtration would wish to say that they could not design the down-stream portion of such a dam in a way to permit the water filtering through the dam to come to the surface without carrying earth with it. There are many large natural springs at the base of sand or gravel banks, and large quantities of water filtering into artificial excavations, which furnish examples of water coming to the surface without disturbing the material through which it comes.

While referring to the percolation through such a dam, it may be well to call attention to the comparative sections of permeable material of the Bohio Dam and the North Dike, as illustrated by Mr. Morison in Fig. 8.*

The area of this material at the North Dike as shown by him is correct, and it is larger than the area given for the Bohio Dam. The degree of fineness of the material, however, is very different. He states that the effective diameters of certain samples from the Bohio Dam average by their squares 1.11 mm. Making allowance for these

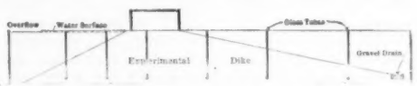

Fig. 3. Experimental Dike Used in Studying Designs for the Wachusett Reservoir.

samples being coarser than the average sample of permeable material and for the fine material lost, he suggests that the average diameter of the material in place may be taken at 0.50 mm. The material classed as "permeable" at the North Dike has an effective diameter of about 0.20 mm., and, since the percolation varies as the square of the diameter, the percolation at the North Dike, with the same loss of head, would be less than one-sixth of the percolation per unit of area at the Bohio Dam.

The design of the North Dike was prepared after consultation with Joseph P. Davis and A. Fteley, Members Am. Soc. C. E., and Mr. Hiram F. Mills, A. M., C. E., of Lowell, Mass., and the final design received the approval of these engineers. Thomas F. Richardson, M. Am. Soc. C. E., Engineer of the Dam and Aqueduct Department of the Metropolitan Water-Works, made most of the preliminary investigations, and Hiram A. Miller, M. Am. Soc. C. E., Engineer of the Reservoir Department, completed the investigations, worked out the details of the design, and has had charge of the execution of the work.

THE UTILIZATION OF THE HEAT OF SLAG FOR DEVELOPING POWER.

By A. W. Clapp.*

The Mitchell-Copeland Power Co., recently incorporated in Los Angeles, Cal., will erect a large power plant at Butte, Mont., using the waste heat of slag from the smelters. By the power thus derived the company proposes to operate the street railway and lighting system of the city.

This company controls patents for utilizing the waste heat of slag now disregarded at smelters and large iron and glass furnace plants.

For example, the smelters at Jerome, N. M., handle more than 900 tons of ore a day, throwing away about 600 tons of slag every 24 hours. Mr. Mitchell claims his new patent will enable 8,000 HP. to be developed from this source.

This system is now in operation at the Cananea (N. M.) smelters, where its efficiency has been demonstrated and practical results have been obtained.

Mr. Mitchell has made an offer, which has been accepted, to work the slag at Butte, where he will furnish the copper plant with free power, and with the surplus he will light the city and operate the street railway system.

*Not reproduced here; see Proceedings Am. Soc. C. E., Jan., 1902.—Ed.

*Salt Lake City, Utah.

SUBAQUEOUS TUNNELLING THROUGH GRAVEL.

At a recent meeting of the Institution of Civil Engineers, in London, Mr. A. H. Haigh, M. Inst. C. E., presented a paper on Subaqueous Tunnelling through the Thames Gravel on the Baker Street & Waterloo Railway.

In passing under the Thames the tunnel left the London clay and encountered a bed of clean gravel and sand lying in an abrupt depression of the clay surface. As this deposit was water-bearing the use of compressed air was necessary. The two tunnels were parallel for a short distance under the river; and then one tunnel rose at a grade of 1 in 111, and the other fell at the rate of 1 in 107, until one tunnel was vertically under the other. Near the Thames Embankment and parallel with it two stages were built, and from these two 16-ft. working shafts were sunk.

The shield for tunnelling through the gravel was provided with a hood in front and a fountain trap behind to afford an air-seal against a horizontal water surface, in case of a run of water. Under the forward top screen of this trap the workmen could escape in case of accident. Laterally the shield was divided into two halves, ahead of the trap, by a vertical girder, which allowed half the face to be attacked, without disturbing the other half, after any accident to the planking due to a "blow." The remainder of the shield was a steel cylinder stiffened by a circular box girder in the rear of the hood, and behind that a strong ring of cast-iron carrying 14 hydraulic rams. In the tail of the cylinder the tunnel rings were built.

The iron lining for the 12-ft. tunnels was made of rings 18 ins. wide. The tunnel was constructed with a continuous longitudinal lining joint, machine-faced and dressed with a mixture of red chalk and Stockholm tar before erection. Joint grooves were calked with iron-rust cement, and grummetted where necessary. Through the tapped holes in the iron lining blue lias lime grouting was forced outside the lining and the holes were then screw-plugged.

Tunnelling began with 17 ft. of solid clay and 34 ft. of total cover beneath the river bed. Before reaching the gravel an 8-ft. solid, air-tight diaphragm wall of brick was built across the tunnel, and provided with working and emergency air locks and necessary pipes. The use of compressed air was begun when the depth of clay over the shield was reduced to 5 ft. As soon as the cutting edge reached within a few inches of the gravel, pockets of pugged clay were made, close to each other, in advance of the face and forming a soft bed for the shield to enter. As the amount of gravel increased the amount of this clay pugging increased. The advance face was timbered with close horizontal planking, against a thick plaster of clay, and arranged in two halves. This planking was first held against the shield, and later by vertical timbers supported by round steel struts passing through the shield when the latter was driven forward. The normal progress of the tunnel in this gravel was three 18-in. rings per day, and three 8-hour gangs were employed. The air-pressure varied with the tide, from 24 to 32 lbs. per sq. in.

Cases of sickness from compressed air were few. The amount of air, at atmospheric pressure, pumped through the tunnel varied between 30,000 and 200,000 cu. ft. per hour; and the proportion of carbonic acid in the air of the working chamber varied from 0.06 to 0.10%.

In driving the east tunnel later, a slight modification was made in this shield to enable the miners to work conveniently with the working face altogether under the hood. An average progress of over 5 ft. per working day was uniformly maintained, while the amount of air delivered averaged only 100,000 cu. ft. per hour.

IRON ORE DEPOSITS IN SWEDEN are to be worked by the Dunderland Iron Ore Co., with a capital of $10,000,000. This English company proposes to use the Thos. A. Edison process for working the mines. Mr. Edison and Lord Kelvin are the technical advisers.

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ADVERTISING RATES: 20 cents a line. "Want" and "For Sale" notices, special rates, see page XXIV-XXIX. New copy for standing advertisements should be received one week in advance of publication; new advertisements, Monday morning. The pages containing "Want," "For Sale" and "Proposal" advertisements are held open until Wednesday noon.

The proposed regulating tanks for storm water from city streets, described in a note elsewhere in this issue, seem particularly worthy of consideration where no provision is to be made for all the storm flow, but where there is danger of water pollution from the first washings of the streets during a rainfall. Under ordinary conditions, such as a separate system of sanitary and storm sewers, it is doubtful whether it would often be necessary, or even worth while, to put in the somewhat complicated and expensive system of regulating tanks and sewer connections advocated by Mr. Richert. It is true, as he suggests, that the regulating tanks turn as much or little as may be desired of the earliest and dirtiest street washings in with the house sewage, and keep all the latter from the stream until the final outlet is reached, whereas the ordinary storm overflows discharge house sewage diluted with more or less dirty street washings. But in most well-designed sewerage systems the combination of diluted sewage and heavy stream flow, the latter providing still further dilution, is sufficient to prevent a nuisance. When such is not the case, would it not be better to use the limited combined system, rather than the double or full separate, and dispose of the excess storm water by way of the gutters? This is Mr. Richert's first and simplest plan, and it is questionable whether it would be advisable to go beyond it, unless under exceptional circumstances.

On another page of this issue appears an article by Mr. Daniel B. Luten, on the design and construction of a concrete-steel bridge of short span. Engineers in general will, we are certain, read this article with great interest, and that for two different reasons: In the first place, because it gives in detail the method of calculating a structure in concrete-steel, and describes how the design was actually executed; in the second place, because it presents a novel method of designing and calculating small arches, a method which is simple and direct, and, which, though not following in the path of rigid mathematical analysis, yet yields results that are known to be both closely correct and closely economical, if the latter phrase may be used.

There is a further feature of interest connected with Mr. Luten's design; the use of steel tie-rods for the arch, and the method employed of imbedding them in a concrete pavement. While a distinct innovation in spirit and method, this feature is of but limited interest compared with the other points raised; it is advantageous, or rather necessary, only in case of small arches, where even slight motion of the abutments would result in serious cracks in the arch, and it can be used only in flat, shallow stream-beds. In the case in question, however, the use of the tierods and the concrete pavement represents a perfectly successful use of simple means to greatly increase the stability of both the arch itself and the foundations of the bridge.

Arches of masonry, and even arches of concrete, are far older than works in concrete and steel, yet, like the latter, they have associated with them many elements of obscurity. Many mathematicians and many engineers have analyzed the conditions to which arches are subject, and their behavior under those conditions; to the general engineer, however, masonry arches are still a matter rather to be avoided, in any case to be approached with caution. About all that has been concluded in the matter up to the present is that some of the main factors entering into their calculation cannot be defined precisely, and that any analysis is at best an approximation. Yet no method has been brought forward for such approximation that would be sufficiently simple and direct to make it of value in application to practical problems encountered by the engineer. The method of design and calculation developed by Mr. Luten is in this connection of much interest. It is both simple and comprehensive, and will, therefore, be found of ready application to many problems in arch-construction. It is a departure from conventional lines in that it determines the outlines of the arch as a result of the calculation, instead of assuming them, and then investigating the stability of the structure. Finally, it is based on a course of reasoning of such nature that the designer may at all times feel full security as to the value of the results he has reached.

ON SEEKING ENGINEERING ADVICE BY CIRCULAR LETTERS INSTEAD OF EMPLOYING AN ENGINEER.

Among the numerous demands upon municipal engineers few are more perplexing than the lists of questions from other cities requesting information. To answer all these questions in detail would be impracticable; to answer them briefly would often be misleading; to ignore them entirely is a practice few would like to follow. What, then, shall be done? Such a question, of course, can never be answered once for all, but a few tentative suggestions may be offered. First, however, we ask those interested to read the communication below which has suggested to us the discussion of this question:

Sir: Knowing the Engineering News to be interested in all matters pertaining to the profession, I desire to call your attention to the enclosed printed sheets which have lately been received at this office. These are but a sample of many inquiries that are constantly arriving, that would keep the city engineer or his assistants busy furnishing gratuitous professional advice to people of whom it may be said that it is very doubtful whether they know enough to use it when they receive it. Is there any other profession so blessed (?) with numerous inquiries from people looking for something for nothing, covering every detail of construction from placing tile drains to building suspension bridges? What would a physician think of a printed circular letter somewhat like this: "Do you have any cases of typhoid fever in your city? If so, state your methods of treatment and reasons therefor," and continued down through the fifteen principal diseases found in practice in a city?

Is it not time that the engineers take a stand to protect those of the profession who are in the consulting [illegible] and with a big blue pencil write across the face of such documents: "You are badly in need of the services of a consulting engineer—see late issues of Engineering News for addresses," and return them to the senders. Would like to further inquire, of what use detailed answers to the enclosed questions from 100 of the largest cities in the country would be to one who is not a professional engineer and has not the ability to sift, digest and assimilate the valuable information they might contain?

It is generally the smaller cities and towns of from 5,000 to 20,000 inhabitants who are looking for engineering information for nothing, and being unwilling to pay a small fee to one who has made a life study of such matters and is familiar with the locality, they prefer to [illegible] their information.

There are cases where it may be perfectly proper for engineers to give gratuitous information to members of their own profession, in regard to certain special pieces of work; but when it comes to writing out whole pages of general information upon subjects that are already well covered in numerous standard works, it is time to respectfully decline.

The enclosed questions were sent out by the Chairman of a Municipal Committee of a town not a thousand miles from New York, and are not selected with any particular reference to that place, but simply as a good example of numerous others in the same line, and to call attention to an abuse of good nature that should be brought to an end. Very respectfully,

W. H. [illegible]
Assistant City Engineer.

Duluth, Minn., April 25, 1902.

There are two classes of information sought by those who send out such circular letters as our correspondent describes. One class relates to the character and extent of public works or the way they are managed, and the other class is in the nature of opinion and advice. There can be no objection to the first class, provided the questions asked are reasonable in number and can be answered off-hand, or by reference to records kept by any well-governed city; and provided further that the questions are asked and may be answered on the basis of mutual benefit through interchange of opinion by city officials. So long as the second class of information is also in the line of interchange of experience or opinion between municipal officials such inquiries are often warranted. But the exasperating feature of this whole matter is that so many city officials, particularly outside engineering departments, ask for so much information that is already available in statistical or engineering books, and are so very free to request engineering advice, all with no idea of making any return in kind, or, much less, in money.

At the sacrifice of some space and with the risk of repeating what must be an old story to many of our readers, we reprint the inquiries relating to paving which were sent to our correspondent:

I. What do you consider the most desirable pavement to lay on a retail street 60 to 70 ft. between curbs, with easy grades and with a traffic of 4,000 teams per day, not more than 5% of which will run above 3 tons and only a very few as high as 8 or 10 tons; wagons for heavy traffic being equipped with tires from 2½ to 6 ins. in width?

II. What do you consider the strong points for and against each of the four principal kinds of pavements, namely: asphalt, wood, granite, and brick?

III. What do you consider the life of each of the above-named pavements under traffic as above indicated?

IV. What is the price per square yard of each of these kinds of pavement in your city?

V. How many miles of the various kinds have been laid in your city to date?

VI. At what rate per annum are you now laying each of the different kinds?

VII. What is the earth in your streets; sand, gravel, or clay?

VIII. What pavement is most extensively used in your city at the present time?

IX. What is the maintenance per square yard per year?

X. What kind has been most extensively used in the past?

XI. If you are using one kind much more extensively than other kinds, is the reason for its more general use one of preference for that particular pavement, or one of cheapness at which you are able to get it laid?

A. What in your judgment would be the saving in wear and tear on horses, harnesses and vehicles on an asphalt pavement as compared with a stone block pavement?

B. Do you consider the slipperiness of asphalt a serious objection to its use?

C. Do you consider asphalt a suitable pavement from the point of durability to lay on a street such as has been described above, and how would you answer the same question in reference to wood and brick?

Questions IV. to X., inclusive, request statistical information which should be available in printed form in the annual reports of all city engineers or boards of public works. Question XI. likewise, is one of fact, rather than of opinion. Question III., had it been worded so as to request the experience of the various localities catechised would fall in the statistical class, but in its pres-

ent form it must be included with questions I., II., B. and C., all of which are largely and some are wholly, requests for gratuitous professional advice. To make the matter worse, as our correspondent suggests, the opinions requested may be found in technical books which, we may add, may be bought for the sum often expended in postage, stationery and clerk hire, to send out such inquiries.

The second list of questions sent to our correspondent are of a character that, we should surmise, the officials of any city would be glad to answer, particularly if assured that the results of the inquiry, in well-digested form, would be sent to all who gave the desired information. The object is to ascertain what officials or boards have charge of public works, the extent to which heads of departments regularly consult with the mayor, and the best method of administering street, water and sewer departments so they will work in harmony with each other and to the financial advantage of the city. But unobjectionable as most of the questions on this list are, to answer them in detail, and to express an opinion of value, would require quite a bit of time on the part of the officials of the several cities addressed, after which the official which sent out the queries would have to spend much additional time to digest the returns, while all concerned would be going over ground already covered by experienced men in recent books dealing with municipal administration.

The subject brought up by our correspondent is a forcible illustration of the enormous waste of time, money and mental energy by municipal officials, who, instead of laying hold on the experience of others that is already available in the form of books or professional advisers, wearisomely traverse unfamiliar ground and draw conclusions which are bound to be faulty.

Much, and oftentimes all, of this trouble, error, needless expense, and even sacrifice of life, would be saved if the municipality would employ a competent city engineer and see that this department was provided with a good engineering library. Some questions would still remain to be settled by other means, but it would generally be best to refer most of these to specialists, who have the incomparable advantage of knowing the practice in other cities, how to sift the good from the bad, and, most important of all, how to apply the knowledge and experience of themselves and others to the local needs of a given community.

By the foregoing we do not mean to convey the idea that there is never just occasion for a city or city official to send out circular letters of inquiry. On the contrary, this practice, intelligently followed, both in framing questions and summarizing the information secured, might be profitably carried to a much greater extent than it has yet been, provided it was conducted on the basis of interchange of information among all the cities involved in the circular inquiries. Some officials not only recognize this, but ensure even wider publicity for the results of their inquiries by sending them to one or more of the technical journals.

In conclusion, city engineers and other municipal officials would be saved much time and trouble in connection with circular inquiries if (1) more pains were taken to include in municipal reports readily comparable statistics and brief descriptions of practice and results; and (2) if those who frame circular inquiries would (a) first make sure that the information desired was not already available in printed form, or not best obtainable from a consulting engineer, and then (b) frame their inquiries so they would relate to the nature and extent of public works, and the satisfaction given by specific materials or methods in the particular locality addressed, and avoid all requests for technical advice.

LETTERS TO THE EDITOR.

The "Leg Bridge."

Sir: My attention was attracted by Mr. Luten's article on "Defective Bridge Construction in the Prairie States" in your issue of April 17.

I wish to say that while all bridge engineers are aware of the theoretic defects of the leg bridge, there is a strong demand for them, and we will undoubtedly build them as long as the demand continues. They are usually chosen, not for really economical reasons, but on account of their lower first cost, where the funds are limited and a great number of bridges are required.

While a probable expansion of ½-in. in the length of a 60-ft. bedstead truss might seriously affect the calculation of the stresses and the members, it is ludicrous to think that two earth embankments will not give enough to allow 20-ft. steel joists to expand ⅛-in., 1-16-in. at each fill. If the joists and legs are properly proportioned at the beginning, what matter if the bottoms of the legs are pushed in a few inches? It will not cause any additional or indeterminate stress in either the legs themselves or the joists.

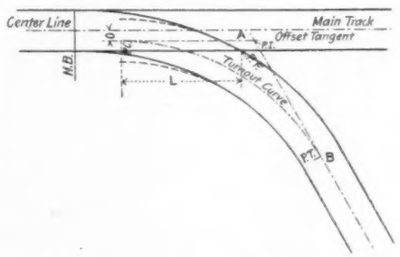
I prefer to proportion the legs for a fluid pressure through their entire length and connect the joists to a cap over the top of the legs without knee braces. This permits the slight movement of the legs, and while there may be conditions conceived that will cause the bridge to become lopsided and that it will fall down, the cases where this is true are extremely rare.

Perhaps when the gentleman has succeeded in convincing the public that a certain concrete arch with wooden ties in the bed of the stream is more enduring and more economical and satisfactory, the counties will be rich enough to replace all their leg bridges with the arches referred to, and we will then be obliged to stop the building of leg bridges.

Yours truly,
April 24, 1902. A Bridge Builder.

Using the Run of the Crusher for Macadamizing.

Sir: Being interested in a large crushing plant, I received a letter or warning of State Commissioner of Public Roads Henry I. Budd, Trenton, N. J., to be careful about the sizes of crushed stone to be used for county roads. Mr. Budd insisted on very uniform sizes for the different layers of the macadam roads, which means, say for bottom course of 8-in. macadam road, have all stones nearly alike, size 2½ ins. The same holds good with the course of 1½-in. size; no 1¾-in. and no 1¼-in. is allowed. Mr. Budd claims the ravelling of roads is largely due to the stone being of such unequal sizes that they cannot properly angle together.



Now, I differ entirely, and I claim the best bottom layer for an 8-in. macadam road should be the run of the crusher which passes through a 3-in. ring, taking out the fine dust or leaving it in; if stones are cheap, leave it in; if stones are more expensive, take it out and put on more binder. For the second course take the run of the crusher to pass through a 2-in., 1¾-in. or 1½-in. ring, according to the thickness of the layer of the second course, taking out only what will pass through a 1½-in. ring and top dress with the ½-in. screenings.

These stones can be handled cheaper by the crushermen, and I am fully convinced pack much better, do not ravel and take less rolling. I would here call your attention to the specifications for large concrete work. Formerly they also required uniform sizes, and now they specify the run of the crusher.

I supplied the stones for about 1½ miles of 8-in. macadam in Bound Brook, N. J., and supplied the sizes of stone which I advocate. As soon as the stones were dumped they packed down at once, the teams had no trouble to go over the newly-spread stone, and the road is in a first-class shape and did not ravel.

I would be pleased to have others discuss this question.

Wm. Harlig.

Bound Brook, N. J., April 28, 1902.

Temperature Stresses in "Bedstead Truss" Bridges.

Sir: In your issue of April 17, in the article on "Defective Bridge Construction in the Prairie States," the author has a certain paragraph descriptive of the "bedstead truss," in which he says:

> The lower chord is usually made a compression piece in the two end panels only, since the pressure of the earth filling against the legs may be assumed to overcome the tension in those two members.

Will you kindly advise me how any tension strains can enter into the members in question in such a truss due to the usual conditions of loading?

Very truly yours, F. W. Mettler.
c/o Wellman-Seaver-Morgan Engineering Co.,
Cleveland, O., April 19, 1902.

(We referred the above letter to the author of the article in question, and append his reply, as follows.—Ed.)

Sir: Referring to the accompanying sketch of a "bedstead truss' bridge with girder approaches instead of the usual earth fill against the backing, it will readily be seen that if the bridge were erected at, say, a temperature of 60° F., and were subsequently subjected to a fall of temperature of 75°, the mudsills, of course, remaining fixed, forces would be induced as shown by the arrows on the right. In the illustration which accompanied the article in question the height of end posts was 5 ft., with legs projecting 8 ft. below. In that structure, if the end posts be assumed rigid, a 75° fall of temperature would induce stresses somewhere between 5-13 × 28,000,000 × 75 × 0.000,005 = 5,250 lbs. per sq. in., and ⅝ × 28,000,000 × 75 × 0.000,006,5 = 8,531 lbs. per sq. in. (where coefficient of elasticity = 28,000,000 lbs. per sq. in., and coefficient of expansion = 0.000,006,5 for 1° F.). These stresses would be distributed between the upper chord in compression and the lower chord in tension. The end posts not being rigid, however, the above stresses would be distributed among the upper and lower chords and the end posts in an inverse proportion to the areas of section of the former and the moments of inertia of the latter. Since the area of section of the lower chord in the end panels will usually be small in comparison with the area of upper chord and moment of inertia of legs, the lower chord in end panels will be called upon to carry most of this tension, which, as shown above, may be considerable even in a truss of two panels only. When the truss consists of more than two panels, the light sections in the end panels would be additionally stressed by the contraction of the heavier chord sections in the middle panels.

While an analysis for the stresses induced by vertical loads alone would require no member in the lower chord of end panels of the type shown, the fact that the structure is in unstable equilibrium would require their presence. Fall of temperature would necessitate their being tension members, while rise of temperature would not necessitate compression members. When this type of bridge is erected with earth filling back of the legs, the lower chord is usually made a compression piece in the two end panels only, since the pressure of the earth filling against the legs may be assumed to overcome the tension in those members.

Very truly yours,
Daniel B. Luten.

Indianapolis, Ind., April 26, 1902.

Laying Out Turnouts.

Sir: After reading the article by Mr. C. M. Kurtz, in your issue of April 3, the writer offers the following method for laying out turnouts. He has used it for several years; it probably has a wider range of application than the method referred to above, and with the use of the table, it requires no further calculation in the field.

Let the figure represent a turnout curve which is compounded at the point of frog; that is, when the main curve is of a different degree than the lead. If the center line of the main curve is produced back to end in a tangent parallel to the main track, the new tangent will be at a distance which we will call O from the center line of the main track.

Let O = the offset as above stated.
L = the length along the main track from a point opposite the P. C. to the point of frog.
F = the frog angle.
G = the gage, and
R = radius of main curve: i. e., the radius of the turnout curve beyond the frog.

$$\text{Then } O = G - (R + \frac{G}{2}) \text{ Vers F. and}$$

$$L = (R + \frac{G}{2}) \text{ Sin F.}$$

Now, if the P. C. is taken on the offset tangent and a simple curve of the radius R run in, the outside rail will evidently cross the main track rail at the required frog angle F, and at the distance L from the P. C. Therefore the turnout can be calculated and run in as a simple curve.

The frog is located by the distance L from the P. C. and the head block is set the length of lead wanted back of the frog. This makes the problem independent of the length of lead about which there is so much diversity of opinion.

This method is adapted to all turnout problems when the degree of curve is known or assumed and it is desired to find the position of the frog, by simply considering the offset tangent in place of the real center line of the main track. The table gives the value of "O" and "L" for all the combinations of degree of curve and number of frog, from 4° to 32° curves and from No. 5 to No. 10 frogs.

Table Showing Value of O and L for Laying Out Turnout Curves.

No. of frog.		Degree of Curve. 4°	5°	6°	7°	8°	9°	10°	11°	12°	13°	14°	16°	18°	20°	22°	24°	26°	28°	30°
5	O	...	...	...	...	9.51	7.69	6.64	5.72	4.80	4.12	3.45	2.44	1.66	1.05	0.54	0.12	0.25	0.57	0.84
	L	...	...	...	...	142.2	127.0	113.8	104.3	95.0	88.3	81.6	71.5	63.6	57.6	52.5	48.3	44.6	41.4	38.7
6	O	...	...	...	...	5.21	4.08	3.24	2.51	1.91	1.41	0.98	0.27	0.26	0.72	1.08	1.37	1.63	1.83	2.01
	L	...	...	...	...	119.0	105.7	95.3	86.6	79.5	73.4	68.4	59.8	53.2	48.0	43.6	40.2	37.0	34.7	32.4
7	O	...	...	4.96	3.62	2.54	1.76	1.10	0.60	0.14	0.21	0.55	1.06	1.46	1.77	2.03	2.21	2.44	2.60	2.74
	L	...	...	136.0	116.5	102.1	90.6	81.7	75.3	68.2	63.0	58.6	51.3	45.7	41.2	37.5	34.5	31.8	29.7	27.8
8	O	6.46	4.22	2.74	1.68	0.89	0.26	0.23	0.64	0.97	1.26	1.50	1.90	2.20	2.35	2.74	2.82	2.97	3.08	3.19
	L	178.8	143.0	119.2	102.2	89.5	79.5	71.6	65.2	59.7	55.2	51.4	45.0	40.0	34.8	32.8	30.2	27.9	25.9	24.2
9	O	4.14	2.37	1.19	0.34	0.28	0.78	1.16	1.49	1.75	1.98	2.17	2.48	2.73	2.93	3.09	3.22	3.31	...	...
	L	159.0	127.2	116.2	90.8	79.7	70.7	63.8	58.0	53.3	49.1	45.7	40.1	35.6	32.1	29.2	26.9	24.8	...	...
10	O	2.44	1.02	0.07	0.61	1.17	1.53	1.84	2.10	2.31	2.49	2.66	2.91	3.11	3.27	3.39	3.50	3.59	...	...
	L	143.0	114.3	95.7	81.9	71.6	63.7	57.4	52.2	47.9	44.3	41.1	36.1	32.1	28.9	26.3	24.2	22.3	...	...

The numbers that are below the heavy line in the table are used where the offset is on the same side of the center line of main track as the frog; and those above the heavy line are used when the offset is on the side away from the frog. Of course, when the turnout curve is the same as the lead curve then O = zero and L = the theoretical lead.

For example, with a straight main track: Suppose it is desired to connect the tangent A B with a 10° curve and use a No. 10 frog. Referring to the table under 10° and opposite No. 10 frog is found O = 1.84 ft. and L = 57.4. Now imagine the offset tangent to be 1.84 ft. from the center line and on the same side as this frog, and intersect the given tangent A B with this line at A, which is the P. I. for this curve. By finding the usual tangent distance for a simple 10° curve the P. C. is set from A and the frog set 57.4 from the P. C. The curve is then run in the usual manner from the P. C. to B.

For a similar case with curved main track, the values in the table hold good, but this rule must be observed: If the main track curve and turnout curve are in opposite directions use the sum of the degrees of curve for the degree of curve or argument in the table. When they are in the same direction use the difference.

In Shunk and Nagle's field books are given a method for connecting a curve and a tangent with another curve. It is only necessary to consider the offset center line in place of the real one to apply this method to that problem.

In cases of turnouts to parallel tracks, it is only necessary to correct the distance between tracks by the amount of the offset and then proceed as with simple reverse curves.

A. L. Grandy,
Akron, O. Supervisor Erie R. R.

A Curious Case of Misbehavior of Concrete Piers Built in Freezing Weather.

Sir: During the winter of 1900 and 1901 I had charge, as Assistant Engineer, of the construction of some concrete piers for a bridge over the North Platte River at Ft. Steele, Wyoming, for the Union Pacific R. R. While constructing this work there were some peculiar actions of the concrete in setting, which I record for the benefit of the profession, in the hope that some older and more experienced heads may give us the reasons for these peculiarities, as it is a puzzle to all my engineering acquaintances. I have handled masonry and concrete exclusively for the past five years, and am pretty well acquainted with the actions of concrete under different conditions; but this is the first and only time that I have had such an experience as is related below:

The conditions were as follows: The bridge sets approximately east and west; the river flows north; the concreting was carried forward as rapidly as possible; quite a good deal was placed when the thermometer registered — 30°. All the material, except the cement, was heated warm enough to burn the hand. The concrete was hand-made, of the dry instead of the wet mixture. I have obtained excellent results with both kinds of mixtures, however. The proportions were 1-3-5 during the coldest weather, and 1-3-6 when the thermometer was near zero. The concrete went into the molds steaming hot, after the cold weather set in, and that was during the erection of the five piers. The stone was wheeled to the mortar bed in barrows with perforated bottoms, and after being drenched with water, was dumped into the mortar.

A few days after the first pier was completed it was tested for alinement on its longer axis, and it was found that the south end of the pier had moved west at the top a half-inch or more." This was quite a surprise to all, as the pier had been tested as erected to keep it true and had been tested after the placing of the last batch of concrete.

Thoroughly on our guard now, we procured good carpenters and framed the frames of the molds upon the ground for the remainder of the piers and raised them into place with a light block and tackle. We were convinced that we had either let the pier slip, or that we had not used care enough in making the frames, and that strains introduced in the frames caused this movement towards the west. With all our care, however, we found that each pier swung west, and that in this instance it was impossible to eliminate the movement of the south end of the pier around the north end as an axis in every instance towards the west. The maximum distance moved was 1¼ ins.; the minimum was a little over ⅝-in.

The heights of the piers had been accurately determined with a Y-level, and the top of the coping had been finished under a straight edge as true as a planed surface. But it was found upon checking the elevations after the concrete had set that the south end of the pier had another surprise in store for us, as it had settled more than the north, ranging from scarcely ¼-in. to a little over ⅝-in. This difference in elevation had not corrected itself four months after the completion of the piers.

It might be added that the sun shines every day in this vicinity with scarcely a break, and that there had been two very severe snowstorms, lasting about a day each, while we were doing this work. The frames for the molds were made of old Oregon fir stringers 8 × 32 ins., split. The molds were of 2-in. unplaned plank. The neat dimensions of the piers are 6 × 16 ft., with a batter of 1 in 24 on all lines. The plaster or skin coat was taken from the regularly mixed mortar for concrete, and at first was applied to the molds with a trowel, just as high as the following course of concrete, which was placed almost as rapidly as the plaster was put upon the forms. This coat was afterwards made by spading next to the forms while the concrete was being tamped. Good results were secured each way.

I find that good results can be procured in concrete work when you have a good, reliable foreman who takes pride in his work. I have never had any trouble, either, with masons not doing as they were wanted, but have always told them why it was necessary to do the work my way when it was different from their accustomed way of doing.

The concrete was placed as rapidly as it was possible to make it, in well tamped courses, from 6 to 10 ins. thick, the thickness of the courses depending upon the amount the concrete mixers had ahead. The last course of the day was made very rough upon top, so the following course would have a good opportunity to bond properly.

With the usual difficulties attendant upon going under water for the foundations, there is little to add. The superstructure consists of four spans of 102 ft. 6 ins. plate girders, with 35-ft. plate girder approaches. The placing of the eight girders was done in less than six hours actual labor (upon two days), and this includes picking up the girders and placing them upon the cars before moving them at a siding nearly a mile distant from the bridge. One of the girders was put into place upon its bed plates in 17 minutes.

W. H. Kutz.
Room 23, R. R. Building, Denver Colo.,
April 12, 1902.

BOOK REVIEWS.

WATER AND WATER SUPPLIES.—By John C. Thresh, D. Sc., M.D., D.P.H., Medical Officer of Health to the Essex County Council, Lecturer on Public Health, etc. Third edition, revised and enlarged. Cloth; 5 × 8 ins.; pp. 527; tables and illustrations. Philadelphia: P. Blakiston's Son & Co. $2, net.

The second edition of this book was favorably reviewed in our issue of July 19, 1900, in which we stated that it was a good elementary treatise, written from the standpoint of an English medical county health officer. The present edition has been expanded by the addition of some 75 pages, apparently fairly well distributed throughout the book. There are two new chapters, on the protection of underground and surface water supplies, respectively.

THE BACTERIAL PURIFICATION OF SEWAGE.—Being a Practical Account of the Various Modern Biological Methods of Purifying Sewage. By Sidney Barwise, M. D., County Medical Officer of Derbyshire. Cloth; 6 × 9 ins.; pp. 48; tables and 12 illustrations. London: Crosby, Lockwood & Son. New York: D. Van Nostrand Co. $2.50.

Perhaps the nature of this book can best be described in a few words by saying that it "is practically the substance of a report" by the author for the instruction of the various district councils of the County of Derbyshire, England, and that it is designed, also to supplement the author's "The Purification of Sewage," published some two years ago. The present volume is valuable in that it concisely sets forth the status of bacterial methods of sewage purification in 1901 in its relation to the problems of quite small towns or villages. The conclusions are largely personal views, rather than an expression of the concensus of opinion of the best authorities, as might be expected under the circumstances. But on the whole the conclusions, locally applied, appear to be sound, and certainly they incline less to commit [illegible] than those of many English writers and officials [illegible] under similar conditions. For Americans, the [illegible] will be suggestive rather than specifically instructive, only because few American communities have to face [illegible] problems so common in England. The book [illegible] a number of interesting half-tone views of small [illegible] sewage purification plants and a few reproduc[illegible] of line drawings. The author disclaims padding, [illegible] the term to indicate that he has not been wordy [illegible] sake of making a big book. The publishers, however, have added to 48 pages of text in large type a [illegible] number of pages of book catalogue, and then bou[illegible] volume in thick boards, so that a book for which a [illegible] of $1.00 would be ample, comes on the American ma[illegible] at $2.50, or more than the price of the volume to wh[illegible] this is supplementary.

PLANE SURVEYING.—A Text and Reference Book for the Use of Students in Engineering and for Engineers Generally. By Paul C. Nugent, A. M., C. E., Associate Professor of Civil Engineering, Syracuse University. New York: John Wiley & Sons. Cloth; 5⅜ × 9 ins.; pp. xvi. + 577; illustrated. $3.50.

In clear terms and with an excellent arrangement of topics, the author here treats of the whole subject of plane surveying. Briefly set forth, the sequence of chapter-heads is as follows: Linear measuring instruments and range-poles, taking up the instruments themselves, the measurement of lines and the use and methods of rods; chain surveying; compass and general surveying; the telescope of surveying instruments, its theory and construction, defects, tests and adjustment; leveling, spirit, barometric and trigonometric; transit surveying; the planimeter and slide-rule; topographical surveying; hydrographic surveying; mine surveying; the solar instrument; U. S. land surveys.

Under each of these heads the subject is treated as completely as possible, and the various theorems and formulas are very generally accompanied by the proper deduction and the presentation of special cases. Mathematics unnecessary in the ordinary practice of plane surveying are avoided, and the treatment of the subject is made as simple and plain as possible; in fact, the handling of this whole subject is marked by a strong sense of the useful and practical. The author has aimed to supply a solid foundation by giving the student complete deductions of theorems, leaving the latter to apply these to the solution of varied problems; truthfully remarking—that there is only one way to gain a mathematical knowledge—and that is to work examples. The separate treatment of the telescope of surveying instruments is especially complete and instructive, and is full of practical and valuable information to the surveyor. The excellent chapter on the planimeter and slide-rule is written by Prof. C. W. Crockett, of the Rensselaer Polytechnic Institute; and the lengthy and very complete chapter on mining engineering by Mr. William S. Hall, professor of mining engineering in Lafayette College.

The appendices to this publication cover problems for solution: a description of the cyclotomic transit, with one spindle; U. S. Land Office regulations for the restoration of lost corners and the subdivision of sections; photographic surveying methods and instruments; and a series of tables useful in connection with surveying operations.

THE NILE RESERVOIR DAM AT ASSUAN, AND AFTER. By W. Willcocks, C.M.G., M. Inst. C. E., etc. London: E. & F. N. Spon, Ltd.; New York: Spon & Chamberlain, 12 Cortlandt St. Cloth, 7¼ × 10¼ ins.; pp. 29; illustrated. $2.50.

This is not a detailed technical description of the construction of the Assuan dam; but it is a most able and interesting statement of the reasons for building the dam and a forecast of the great advantages to result from it and from the realization of plans for further storage of the Nile waters. The basin irrigation of Egypt, which has prevailed for 7,000 years, is now rapidly giving way to a system of perennial irrigation; with double annual crops replacing the ancient single crop system. Of the 6,250,000 acres of cultivable land in Egypt, only 1,730,000 acres are now irrigated by the old basin system; the remainder is either receiving perennial irrigation, or is waiting for the creation of the reservoirs that can alone supply this want. It is calculated that Egypt needs for its perfect development 30,000 cu. ft. per second of summer flow in the Nile. Even in poor years the Nile can be depended upon for 8,000 cu. ft. per second, leaving 22,000 cu.ft. to be provided by reservoirs; and this latter is

...d at the canal heads for 75 days. The Assuan dam ...uilt, will provide 70 milliards of cubic feet of water, ...of the 200 milliards needed for the full development of ...; this supply will be sufficient for the whole Nile ...y below Assuan, and will increase the rental value of ... territory by $30,000,000 per annum. The cost of ...ging the basin canals of Upper Egypt into perennial ...ls and completing the drainage system, is estimated ...25,250,000. To this should be added the cost of stor... water in the Nile Valley, $30,000,000, and for the res... at Ancient Moeris, or Wady Rayan, $15,000,000; or ... $80,000,000 in all. But the estimated capitalized ...ft of this is $300,000,000. Mr. Willcocks then gives ...history of the Assuan dam project, and discusses the ...ation in his original plans, from a crest 100 ft. above ... to one 26 ft. lower, as a result of the attempts ...ve the Philae Temple from being flooded. But as ...ied out it will be possible to hold up the water to a ...ght of 93 ft. above the Assuan gage, and impound 70 ...ead of 35 milliards of cubic feet of water. The final ... of the dam in this plan is estimated at $8,750,000. ...is dam is strong enough to withstand a maximum flood ...charge of 475,000 cu. ft. per second at Assuan, and the ...thor believes it marks an epoch in dam building, though ...its use is restricted to broad rock foundations. But for ...arrow gorges Mr. Willcocks has great faith in steel-...concrete construction, and a dam for Assuan was designed ...by M. Hennibique which had much merit. The dam ...as built is 2,000 meters long and perfectly straight, in ac...cordance with the recommendation of the International ...Commission. But Mr. Willcocks considers this a mistake, ...and believes the engineer should have been free to follow ...the line of soundest rock, regardless of "majestic appear...ance," etc., supposed to be inherent in the straight-line ...dam.

The most interesting portions of this work lie in the dis...cussion of the supplementary supplies of water for which we must go to the head of the Nile itself. The Abys...sinian Lake Tsana has a surface of 1,150 sq. miles, and a ...catchment basin of 6,600 sq. miles, and would store at ...least 100 milliards cu. ft., which would come down the ...easy channel of the Blue Nile, to Khartoum and the old ...Nile. But the writer objects to this reservoir on political ...grounds as being at the absolute mercy of the Abyssinian ...Emperor. He turns then to the Victorian and Albert ...Nyanza lakes at the source of the White Nile, which are ...in British territory. Lake Victoria Nyanza has an area ...of 26,000 square miles and a catchment basin of 100,000 ...sq. miles, with a mean annual rainfall of 30 ins. A mean ...discharge of 30,000 cu. ft. per second, all that Egypt ...needs, represents only one-sixth of the total rainfall; but ...this proportion is not too small when we consider the ...great evaporating power of 26,000 sq. miles of water lying ...under the equator. This 30,000 cu. ft. per second would ...pass the Ripon falls on a width of about 400 m. and could ...be dammed with infinite ease. Owing to the fact that the ...lake covers 25% of the total catchment basin, the complete ...closing of the outlet would only raise the lake surface 20 ...ins. per year; so that if an unfriendly power were to ...secure control of this great lake, Egypt might be deprived ...of its summer supply for 10 or 15 years in succession ...without any extraordinary difficulty. Lake Albert Ny...anza has an area of 1,700 sq. miles, and—including that ...of Victoria, a catchment basin of 135,000 sq. miles. The ...annual rainfall is 30 to 35 ins., and the mean annual dis...charge of Lake Albert may be taken at 30,000 cu. ft. per ...second. The regulating works would have to be located ...at the crest of the Fola rapids, 140 miles down stream ...from the lake proper; 140 miles beyond Fola, near Lado, ...the river enters the low flat plains, and at Lado the mean ...discharge can be taken at 40,000 cu. ft. per second. Be...yond Lado the river would probably maintain channels of ...considerable magnitude, were it not for the presence of ..."sudds" or bars of floating vegetation, sometimes as much ...as 15 ft. thick. These bars turn the river out of its ...course and some means would have to be devised for their ...removal.

Tabulated, the distance from Lake Victoria to Assuan is 2,740 miles, and the present mean discharge at the latter point is 100,000 cu. ft. per second, with a minimum of 7,000, and a maximum of 475,000 cu. ft. per second. The rapid diminution of the minimum discharge is the strik...ing feature of the table presented. This minimum is 18,...000 cu. ft. at Lado, 600 miles from Victoria, and 10,000 cu. ft. at Sobat, 1,060 miles down. Mr. Willcocks gives his plan for stopping this enormous waste of much needed water. He would raise the water in Lake Albert 12 ft. by suitable works and thus store 600 milliards cu. ft., and this stored water would only lower Lake Victoria 9 ins. The latter lies above Lake Albert and is admirably adapted as a feeder to the smaller lake which can be more easily utilized. He estimates $7,500,000 as a liberal allow...ance for cost of the controlling works, and thinks they would actually cost much less. Such works would per...mit a minimum discharge of 50,000 cu. ft. per second for five months in the year past Lado, and with this we might expect 20,000 cu. ft. at Assuan, in addition to the storage of the Assuan reservoirs. The remainder of the work is devoted to an interesting history of the little-known Nile between the 10th and 5th parallels of latitude, and plans and estimates for the better development of the Soudan.

KONDENSATION.—Ein Lehr und Handbuch über Kon...densation und alle damit zusammenhängenden Fragen, auch einschliesslich der Wassesrückkühlung. Von F. J. Weiss, Civilingenieur in Basel. Cloth; 6×9 ins.; pp. 384; 96 illustrations; Julius Springer, Berlin, 1901. Price, $4.00.

A condenser is a piece of apparatus designed to accom...plish two objects, (1) to condense a certain amount of vapor per unit of time, and (2) to produce and maintain a "vacuum," or low gaseous pressure, in the space occu...pied by that vapor. Sometimes the latter object is the prime consideration, sometimes both are of equal import...ance. In general, it is desirable to maintain as high a vacuum as possible in the condenser. To maintain this vacuum and also to supply the condensing agent requires a series of pumps; thus, though the object of the con...denser is to absorb excess energy, it requires to accom...plish this a considerable extra input of power. The con...denser in this way becomes a complex mechanism and is subject to theoretical calculation in regard to the working of its parts and their effect on each other.

In view of the fact that the introduction of the separate condenser was the first great impulse in the development of the steam engine, it seems strange that no complete theoretical investigation of condensers has yet appeared. Such is the fact, however, and it may be said that engi...neering literature in English is particularly deficient in this respect. Though European engineers had not, up to the present work, given the subject thorough treatment, some beginnings had been made, and several essays had been published that prepared the way for a full investi...gation.

The present work is an exhaustive treatise on con...densers. The treatment is, as might be expected, mainly theoretical, but it is developed throughout with constant reference to actual condenser practice and the problems arising in connection therewith, a subject with which the author has become familiar through years of work in this field. Though the treatment as developed in this work is applicable to condensers for any use, in steam plants, chemical works, sugar factories, etc., it is carried through with particular reference to their main use, in steam-power plants.

The author divides the subject for separate consideration according to the specific differences in operation of con...densers, into (1) "Mischkondensatoren," which word, un...fortunately, we can translate only by "jet condensers," and (2) "Oberflächenkondensatoren," or surface condens...ers. These two main classes may be subdivided into (A) those with "wet" air pump; and, (B) those with "dry" air pump, according to whether the air and the water are removed from the condenser together or separately. A further important classification is into (a) "Gegenstromkon...densatoren," and (b) "Parallelstromkondensatoren." All kinds of condensers may be grouped according to this classification: there is, however, one kind of condenser which comes under the head "Parallelstrommischkonden...satoren," which is not considered in the present work—the Körting "Strahlkondensator" or injector condenser. This form is, as stated by the author, not amenable to purely theoretical calculation and is therefore not taken up.

It will be of advantage to give here an abstract of the first chapter of the book; this in connection with the titles of the remaining chapters will sufficiently indicate the scope and value of the work.

MISCHKONDENSATION.—The condenser consists of an enclosed space in which the incoming steam is condensed by cold water, and an "air pump" which maintains the vacuum, or low gaseous pressure, in this space. The latter pump may be used also to remove the excess of water ("wet" air pump), but ordinarily, a separate pump or a barometric draft-tube is used for this purpose. The pressure in the condenser is composed of two parts, the pressure of the contained air (derived in part from the air dissolved in the cooling water, but principally leaking in at various points) and the pressure of the water vapor. The vapor is to be condensed, the air requires to be re...moved by the air pump. There will, however, be at all points within the condenser a certain vapor tension cor...responding to the temperature at that point; the air pres...sure varies so as to make the total pressure equal through...out the condensing space. In other words, there is a larger proportion of air in the cooler parts of the con...denser than in the warmer parts. Since the air pump is designed to remove the air only, whereas the water vapor is intended to be condensed, the connection to the air pump should be made at the coldest part of the condenser. The proper arrangement of the apparatus then is to have the steam enter at the bottom; the cold water at the top of the condenser. The air pump is connected near the cold water pipe, while the hot water is drawn off at the bottom. This is the principle of arrangement of the "Gegenstrom" condenser (obviously, the desired advan...tage is lost if the hot water and air, after being drawn off separately at the proper points, pass into the same pump chamber). When the steam and the circulating water enter at the same end and flow through the condenser in the same direction, we have the "Parallelstrom" conden...ser; in this type the air and water may be drawn off sep...arately or together; but they will always be at the tem...perature of the hot water.

Then with equal results, the second type will evidently require a much larger air pump than the first type, and will consume more power in operating the pump. Or, given the same amount of condensing water and same tem...perature rise, the vacuum obtainable with the "Gegen...strom" is higher than that obtainable with the "Parallel...strom" condenser. Moreover, the size of air pump re...quired with the former has a definite maximum which may be calculated, and which gives the vacuum corre...sponding approximately to the temperature of the hot water; with the latter the air pump may be increased in...definitely in size, but it can never produce the vacuum corresponding to the hot water temperature.

The circulating water required in either type, with a given temperature rise, is found by the following easily derived and closely approximate formula:

$$\frac{\text{w't circulating water}}{\text{weight steam}} = \frac{W}{D} = n = \frac{570}{t' - t_0}. \quad (1)$$

where t' and t_0 are the temperatures in centigrade degrees of the hot and the cold water respectively.

The capacity of the air pump is given by the formula:

$$\text{For Gegenstrom,} \quad v_0 = \frac{L}{p_0 - d_{(t_0 + a)}}. \quad (2)$$

$$\text{For Parallelstrom,} \quad v_0 = \frac{L}{p_0 - d_{t'}}. \quad (3)$$

where

L = volume of air entering condenser per unit of time;
v_0 = capacity of air pump in unit of time;
p_0 = condenser pressure (absolute);
d_t = vapor pressure corresponding to temperature t, and $a = 4° + 0.1\ (t' - t_0)$, as determined by experiments.

To give the maximum vacuum for given t', we make $p_0 = d_{t'}$ in equation (2), or for Gegenstrom,

$$\text{max. } v_0 = \frac{L}{d_{t'} - d_{(t_0 + a)}}.$$

For Parallelstrom, equation (3) shows that p_0 must always be greater than $d_{t'}$, and hence no maximum value of v_0 is possible.

The value L is composed of two parts, the air entering with the circulating water, and the air leaking in at cylinders, exhaust piping and condenser; the latter part is much the larger. For the former we may write $l_1 = \lambda W = 0.02\ W$. The second part may be put in the form

$$l_2 = \mu D = \mu \frac{W}{n}.$$

Several experiments give, for steam plants, values of μ from 2.0 to 3.0; with very long exhaust piping this may be several times these amounts. For steam plants, then, we have

$$L = 0.02\ W + (2.0 \text{ to } 3.0)\ \frac{W}{n}. \quad (4)$$

For sugar factories an average value of $\mu = 8.0$ was found; for other cases of condensing plants it should be separately determined or approximated.

Combining (2) or (3) with (4) we may then write for Gegenstrom,

$$v_0 = \frac{0.02 + \frac{\mu}{n}}{p_0 - d_{(t_0 + a)}}\ W. \quad (5)$$

For Parallelstrom,

$$v_0 = \frac{0.02 + \frac{n}{\mu}}{p_0 - d_{t'}}\ W. \quad (6)$$

which shows that the ratio of the required air pump capacity to the circulating water is a function of p_0, the vacuum pressure.

Formula (6) evidently holds for "dry" air pump only; for a "wet" air pump, the required capacity is

$$C = v_0 + {}^{w}V + D = \left[\frac{1 + \frac{0.02 + \frac{\mu}{n}}{p_0 - d_{t'}}}{1 + \frac{1}{n}}\right] W. \quad (7)$$

From equations (6) and (7) we may find, for a given vacuum, given condensing capacity, and given tempera...tures of hot and cold water, the minimum value of v_0 (or $v_0 + W + D$) and the corresponding amounts of circulat...ing water required. On account of the large amounts of the latter (large values of n) required to give this mini...mum, it will in general be desirable to adopt some other relative values of n and v_0.

If the condenser has a "wet" air pump, it will be of in...terest to determine the effect on the vacuum of increasing the amount of circulating water; if the air pump (of ca...pacity C) be run at constant speed. By substituting

$$v_0 = C - (W + D)$$

in equation (6), we get,

$$p_0 = \frac{0.02\ n + \mu}{\frac{C}{D} - (n + 1)} + d_{t'},$$

and by plotting as ordinates the values of this function with varying n, we find that at a given ratio n the con...denser pressure p_0 is a minimum, and approaches infinity for $W = 0$, and $W = C$; in other words, by using too

much circulating water, we "drown" the air pump and the vacuum fails.

Any "Parallelstrommischkondensator" may be fitted with an after-condenser, which latter should be fitted with separate air and condenser pumps. The operation of the condenser can in this way be considerably improved. (The amount of the economy may be calculated; the formulas and considerations applicable are given in this chapter of the book.)

In regard to the time required for condensation of the incoming steam, the following experiment was made: An

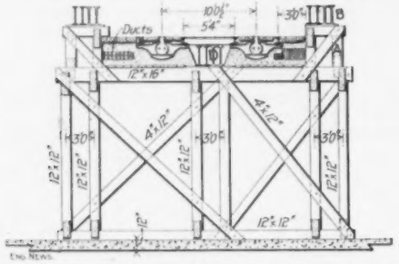


Fig. 33. Trestle Bent Supporting Girders Carrying Street Railway Track.

indicator diagram was taken the indicator cylinder being connected directly to the condenser chamber, and the reducing motion being operated from the engine crosshead. The water was "sprayed" in, or rather it merely flowed in at low velocity through a perforated pipe. It was expected that a rise in pressure would be shown at the ends of the stroke, falling gradually to normal condenser pressure; it was found, however, that the pressure line was a straight horizontal line. The conclusion appears justified, therefore, that even the simplest system of water distribution in the condenser will be sufficient, and that all artifices for finely subdividing the incoming water are superfluous.

A separate section of this chapter is devoted to a description and discussion of the "Weiss Gegenstromkondensator," which was designed in accordance with the principles developed, and which the author has installed in numerous steam plants in Germany. It is shown that when the air pump is run at a capacity exceeding the amount of incoming air, the pump will prime and the vacuum rapidly fall to atmospheric; a special device is introduced in the Weiss condenser to guard against such an occurrence. The author advises that every condensing plant be fitted in addition to the vacuum gage, with two thermometers, . to indicate temperature of incoming and outgoing circulating water. The three readings, together with a simple table, will enable the attendant to judge at all times of the operation of the condensing plant.

The remaining chapters of the book are as follows: 2. Surface Condensation. 3. Starting of Condenser. 4. Power Required. 5. Economy of Condensation in Steam Plants. 6. Calculation of a Condensing Plant. 7. Exhaust-steam Piping. 8. Valve Motion in Condensing Engines. 9. The "Weiss" Valve Motion. 10. Condensation with Varying Steam Consumption. 11. Cooling the Circulating Water (Cooling Towers). A short steam table is appended to the book.

SEMICIRCULAR WOOD STAVE FLUMES.

Flumes of semicircular section and made of wooden staves held together by round-iron binders and a horizontal tie-beam above, have been in use in California for conveying water for hydraulic mining and irrigation purposes, since 1892, and some of them have been in continuous

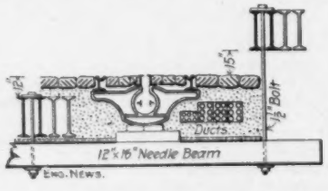


Fig. 35. Detail of Girder and Needle Arrangement.

use since. In an article written for the "Mining and Scientific Press," of San Francisco, by Guy Sterling, C. E., M. E., the inventor of this type of flume, we find some notes on this type of construction that are of interest.

A half-round wooden flume, for 13 sq. ft. of water area in a 16-ft. length, requires 444.5 ft. B. M. of lumber and 96.91 lbs. of metal; a wooden box-flume of the same length and water area would require 646.5 ft. B. M. of lumber and 10 lbs. of metal. But for 13 sq. ft. of water area the ordinary box-flume has 10% less carrying capacity than the half-round flume, by reason of the difference in hydraulic radius. The box-flume also requires 45% more wood—a perishable material—in its construction; and the first cost of this type of flume will be about 10% more than the half-round type. Above 13 sq. ft. of water-area, the differences are greater in favor of the half-round design; while below 13 sq. ft. the difference tends to disappear.

Under fair conditions, an experienced crew of two or three men and a foreman can build from 75 ft. to 150 ft. of half-round flume per day, depending on the size of flume and the alinement. On curved alinement, up to 6 ft. diameter of flume and 80 ft. radius, this flume is built out straight on the tangent to the curve, and is then bent to line. But for a flume as large as this the radius should not be less than 200 ft.; when curves of shorter radius are necessary or unavoidable, the flume can be mitered to fit them.

The best material is California redwood, or Washington cedar; but Oregon fir and Texas pine can be depended upon for good results.

Some of these flumes have been allowed to stand dry for long periods; and afterward the cost of cinching up the bands to put the flume in first-class condition, was only 4 cts. per lineal foot. Under normal conditions, the total cost of maintenance, for flumes of 15 to 30 sq. ft., of water area, has not exceeded 2 cts. per lineal foot per annum.

In setting up these half-round flumes, two outside forms are used and in these the staves are laid, with the ends of adjacent staves breaking joints 2 to 3 ft. With the staves to fill a section in position, the bands, tie-beams and washers are put in place, and the nuts on the ends of the bands are then tightened up, binding the staves together. As the staves are being pulled together they are "rounded out" by beating with a leather-faced mallet, until the inside is smooth and the longitudinal joints tight. Iron tongues, 1½ in. wide, ⅛-in. thick and a little longer than the stave is wide, are next driven into the saw kerfs cut in the ends of the staves for this purpose. Another form is now set up and a second section put in place; with the stave so held that the tongues already inserted in the ends of the old section can enter the grooves in the stave-ends of the new section; and the joints are then closed, by driving the staves with a wooden driving-bar and heavy hammer, until the joints are water-tight.

A detailed article on this round and half-round wooden flume construction will be found in Engineering News, Vol. II., 1896, pp. 354.

STEAM SUPERHEATING as a means of attaining high fuel economy has been adopted to only a limited extent in the United States, while European engineers have made great progress in this field during the past dozen years. The Providence Engineering Works, of Providence, R. I., has recently taken up this matter, however, and informs us that it has now under construction nearly 7,000 HP. of engines designed to use steam superheated to 600° or 700°. Schmidt superheaters are to be used, the Works having taken the American agency for the Schmidt system, and reheaters will be used between the cylinders of compound or triple-expansion engines.

THE SUGAR CONSUMPTION of the United States, the U. S. Bureau of Statistics, has grown from 1,0[illegible] 342 lbs. in 1870, to 5,313,937,840 lbs. in 1901; or [illegible] lbs. to 63 lbs. per capita per year. Of the con[illegible] in 1901 over one-sixth was produced in the Unit[illegible] and about another sixth in our insular possessions. The remaining two-thirds was imported.

THE INTERNAL NAVIGATION OF FRANCE, [illegible] late issue of the "Journal Officiel," includes 3,301 [illegible] of canals, and 4,605 miles of rivers and streams [illegible] for navigation of some kind. The tonnage carried [illegible] the canals in 1901 amounted to 17,017,590 tons; and th[illegible] ried on rivers and streams was 13,350,622.

THE NEW YORK RAPID TRANSIT RAILWAY.

VIII.

Contract Section III.

Contract Section III. of the New York Rapid Transit Railway extends from Great Jones St. to 33d St. The route in detail is as follows: From Great Jones St., Lafayette Place is followed to Astor Place, thence the line curves east under Astor Place and private property to Fourth Ave., which is followed to 33d St. The subcontractors for this section are Holbrook, Cabot & Daly, of New York, whose engineer in charge of the work is Mr. Henry B. Seaman, M. Am. Soc. C. E.

MATERIALS PENETRATED.—The materials penetrated in this section of the subway vary considerably in character. Beginning at Great Jones St., the excavation north to near 11th St. is through a surface layer of rubbish overlying loam and sand. From 11th St. to 15th St., the excavation encounters rock, which in places rises nearly to the street surface, and in other places dips below the foundation bed. From 15th St. north to 18th St., the rock lies only a short distance beneath the street surface, and from 18th St. to 22d St., rock

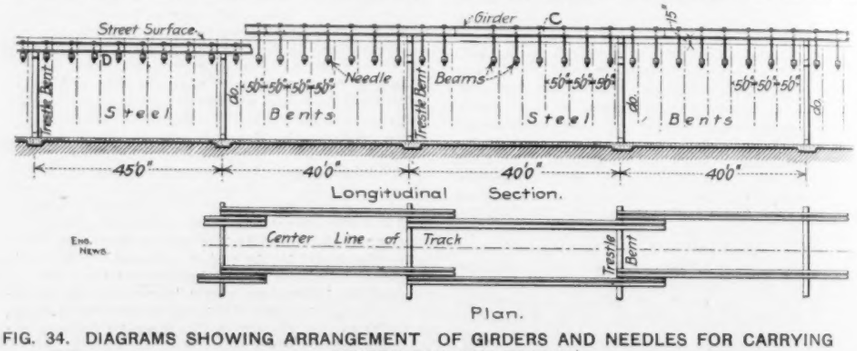


FIG. 34. DIAGRAMS SHOWING ARRANGEMENT OF GIRDERS AND NEEDLES FOR CARRYING STREET RAILWAY TRACK.

bottom, hard pan and gravel are met. For three blocks between 22d and 25th Sts., the excavation is through made ground formed by filling in a former swamp, and considerable water is encountered. North of 25th St. to near 33d St., hard pan and gravel are met, and at 33d St. rock is again penetrated. The rock is a compact mica-schist for the most part, but at intervals a layer of soft rock is encountered which is quite unstable, and is likely to slide. The dip of the strata is from east to west at an angle of about 30°.

DESCRIPTION OF STRUCTURE.—The underground structure throughout Contract Section III. is the standard four-track subway of steel and concrete, with the exception of the stretch from 11th St. to 18th St., where an additional track is laid to provide switching facilities between the local and express tracks north and south of the express station at 14th St. Besides the 14th St. express station there are five local stations located at Eighth, 18th, 23d, 28th and 33d Sts.

METHODS OF WORK.—Several different methods were employed by the contractors for conducting the work on Section III., due to the differences in the character of the material penetrated and of the street traffic to be maintained.

From Great Jones St. to Astor Place there were no surface car tracks to be maintained, and the material penetrated was soft earth, so that open-trench methods were possible. From Great Jones St. to 4th St., a single trench was opened the

whole width of the subway. This trench was sheeted and cross-braced in the usual way, with the pipes supported by chains from the timbering, and the subway structure was built in the open cut. From 4th St. to Astor Place a trench was opened one-half the width of the street in

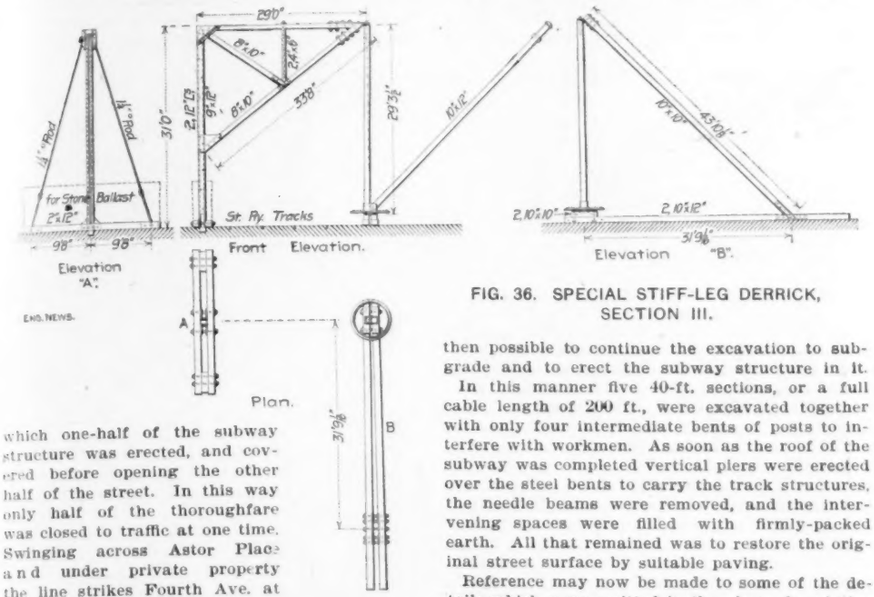


FIG. 36. SPECIAL STIFF-LEG DERRICK, SECTION III.

which one-half of the subway structure was erected, and covered before opening the other half of the street. In this way only half of the thoroughfare was closed to traffic at one time. Swinging across Astor Place and under private property the line strikes Fourth Ave. at about 9th St. The method of construction for this section has not been decided upon.

The methods of work adopted along Fourth Ave., from 9th St. to 33d St., had to be such as would not interfere with the traffic of a double-track conduit electric railway during construction. To do this the tracks and conduits had either to be shifted to one side of the street during excavation and then rebuilt after the subway was completed or else they had to be underpinned and supported in place so that excavation could go on underneath them and the subway structure be built. The last solution was the one generally adopted. It is illustrated by Figs. 33 to 35, inclusive.

Briefly described, trenches are sunk transversely across the street beneath the car tracks at intervals of 40 ft., and in soft ground were carried down to subgrade of the subway. At the bottoms of these trenches were placed footings of concrete, and on these were built timber bents of 12 × 12-in. posts and 12 × 16-in. caps. The resulting construction was a series of trestle bents 40 ft. apart, as shown by Figs. 33 and 34. At the ends of each trestle bent there was erected a pony bent, consisting of two short vertical struts, with a cap piece, as shown at A A, Fig. 33. A pair of 24-in. 80-lb. I-beams were then placed on each side of the tracks so as to span the 40-ft. space between bents. These girders are shown at B B, Fig. 33, and in side elevation at C and D, Fig. 34. They were 50 ft. long, and were so placed as to be flush with one bent and to overlap the other about 10 ft., as indicated by the sketch plan in Fig. 34. The next step was to excavate a trench between the tracks and place the I-beams shown at D, in Fig. 33. This trench was afterwards roofed over by a platform consisting of 6 × 6-in. cross timbers, spaced about 3 ft. apart c. to c., and carrying a floor of 4-in. strips.

As soon as the bents and girders were in place work was begun sinking trenches on each side of the car tracks and driving headings under the tracks from these side trenches. These headings went down some seven feet below the caps of the trestle bents, and as they progressed vertical struts were inserted to carry the track structures. When the headings had been completed and the tracks strutted for the whole 40 ft. between two bents, 12 × 16-in. timbers were inserted underneath the tracks and transversely across the street, and were hung from the side and center I-beam girders by threaded bolts, as indicated by Fig. 35. These transverse timbers or needles were spaced about 5 ft. apart, as shown by Fig. 34. When all the needles were in position the threaded bolts were tightened up gradually until the weight of the track structures was taken off the strutting and transferred to the three double I-beam girders, and through them to the trestle bents. It was then possible to continue the excavation to subgrade and to erect the subway structure in it.

In this manner five 40-ft. sections, or a full cable length of 200 ft., were excavated together with only four intermediate bents of posts to interfere with workmen. As soon as the roof of the subway was completed vertical piers were erected over the steel bents to carry the track structures, the needle beams were removed, and the intervening spaces were filled with firmly-packed earth. All that remained was to restore the original street surface by suitable paving.

Reference may now be made to some of the details which were omitted in the above description to avoid confusion. In excavating the parallel side trenches cableways were used to handle the excavated material. Two cableways were used, one on each side of the street. In some instances the material was handled by stiff-leg derricks. A pit was sunk the width of the trench, and from this pit the trench was excavated along the street by gradually breaking down a vertical face, using narrow-gage tracks and push cars running along the bottom of the excavation to take the material from the vertical face to the derrick. Mention was made of the overlap of the girders carried by the trestle bents. The purpose of this overlap was to permit the bent to be shifted ahead when the adjacent floor was finished, so that construction could proceed in the place formerly occupied by the bent. Sometimes in hard ground the sinking of shafts for the posts of the bents was not necessary, since it was possible to introduce longer and longer struts as the excavation went down. In a few instances large timbers were used in place of the I-beam girders, but this was only when there was a delay in securing the steel beams. It was found that these wooden beams cost nearly as much as the steel I-beams, while their scrap value was practically nil as compared with the steel beams. It is, perhaps, worth noting that these 24-in., 50-ft. steel beams were not infrequently fractured by the shock of unloading.

Previous to the development of the double cableway method just described, a similar method was employed working from one trench and opening only half of the total width of the subway excavation at once. The mode of procedure was as follows: At intervals of about 40 ft. drifts were extended transversely across the street underneath the tracks to a point a little beyond the center of the subway and down to subgrade. The bottoms of these drifts were covered with a layer of concrete, on which was erected a trestle bent, the cap piece of which was just below the railway conduit structure. A shallow trench was then opened between the tracks and a double I-beam girder was placed in it so as to span the 40-ft. opening between two trestle bents with an overlap of 10 ft. This trench was then roofed over by a timber platform. Next another double I-beam girder spanning the 40-ft. space between bents, was placed outside of the track. The two adjacent pits were then connected by a trench, using cableways or derricks and push cars for hoisting out the material. When this trench had been sunk deep enough to give headroom under the surface track structure narrow transverse drifts were driven under the tracks, and in them was set a row of vertical struts. These rows of struts were about 5 ft. apart, and when the material between them was all removed they carried the track structure between the trestle bents. The next step was to place 12 × 16-in. × 15-ft. needle beams between each two rows of struts and suspend them by screw bolts from the parallel I-beam girders. After all the needles had been placed the screw bolts were tightened until all the weight of the track structure had been lifted from the rows of props and transferred to the I-beam girders. The excavation, including the side trench, was then completed to subgrade and transversely to a point just beyond the center line of the subway, and

FIG. 37. VIEW SHOWING EXCAVATION AND TEMPORARY PLATFORM FOR STREET TRAFFIC ALONG EAST SIDE OF UNION SQUARE.

the sides were sheeted and cross-braced. One-half of the subway structure was then erected and the backfilling and paving completed. The second half of the street was then opened up and the remainder of the subway structure was erected and connected up with the half first built.

Before passing on some description of the special derrick used where only one trench was allowed seems called for by the novelty of its design. The general details of this derrick are shown by Fig. 36 and its construction as well as the conditions which prompted its design are outlined as follows by Mr. Henry B. Seaman, M. Am. Soc. C. E., the engineer for the contractors:

When this contract was commenced we were required to keep open for traffic, one-half of the street, while we did our excavating on the other half, and as a double track electric railroad goes up the middle of Fourth Ave., it was found advisable to construct a stiff-leg derrick, with one stiff leg spanning the two tracks and the other parallel to them. The stiff leg parallel to the two tracks formed a complete triangle carrying a bolster on the horizontal timbers so that the whole could be lifted on rollers and moved to a new position quickly. The stiff leg which spanned the two tracks was made of a vertical leg, trussed over the two tracks to the top of the mast. The vertical leg was strengthened with channel beams so as to withstand the transverse strain. It was let down into the paving blocks 8 ins., and was held by about eight tons of iron, or stone ballast, which was so arranged that it could be lifted on rollers and moved back as the work progressed, simultaneously with the stiff leg parallel to the tracks. The use of this vertical leg kept the drive-way on the side of the street opposite the excavation, practically clear for wagon traffic. Six of these derricks were constructed and were in general use until we obtained permission to open up both sides of the street simultaneously, when they were laid aside and twin cableways were used instead.

Roughly speaking, the double cableway method of underpinning the tracks was adopted from 17th St. to 33d St., and the single-track method was used between 9th and 14th Sts. From 14th St. to 17th St. Fourth Ave. skirts the east side of Union Square. Here a different method of excavation was adopted, as shown by Fig. 37. The electric conduit railway tracks were moved over to the east side of the street and rebuilt. A trench was then opened up on the west side of the street wide enough to permit three tracks of the subway structure to be built. The method of rebuilding the surface tracks on the east side of the street was to construct a timber platform about 3 ft. above the original street surface, and to set the conduit yokes underneath it. This platform was fenced in on both sides and formed a temporary street surface for all traffic. At cross streets bridges were carried across the open trench, as shown by Fig. 37, and the platform was connected by inclined platforms with the side streets. The manner of opening the trench on the west side of the street is shown by Fig. 38. As will be observed, a narrow trench on the plane of the west subway wall was opened by sinking a pit and enlarging it to a trench by working the vertical faces up and down the street. This trench was then widened by breaking down the inside wall of the trench in thin sheets which were torn down by blasts. The view, Fig. 36, shows a length of trench, the arrangement of the drill holes and various other features of the plant and work. When the trench had been made wide enough for three tracks this much of the subway structure was built, the electric railway tracks returned to their original position, the backfilling and paving completed, and the east side of the street opened up for building the remaining two tracks of the subway structure. While the section from 14th St. to 17th St. was built as just described, seams of rotten rock were unexpectedly encountered which permitted the sides of the street on which the track was temporarily located to slide, and the track was later partly carried on temporary trestles.

The methods of hoisting the earth and rock from the trenches, as already stated, was by cableways and derricks. The excavated material, except that needed for backfilling, which was spoiled on the adjacent ground, was removed by wagons. Various kinds of wagons were employed, but particular success was had with the Shadbolt dumping wagon, made by the Shadbolt Mfg. Co., of Brooklyn, N. Y. A sectional elevation of this wagon is given by Fig. 39, and its construction and operation are explained as follows by the manufacturers:

All dumping wagons heretofore built have one or two serious defects. Either the weight is carried entirely on the hind axle or nearly so, thereby adding very largely to the difficulty of draft; or rollers or some complicated machinery are used to shift the main body back on the supporting frame, so as to cause the center of gravity to come back of the axle and allow the body to tilt.

The outline drawing shows the construction of our dumping wagon, the body being balanced on the hind springs, so that it is easily tilted without the use of any mechanism, the springs resting on a bar across the frame. This bar pivots in steel sockets and turns with the body. The sockets in which the bar turns are set in the frame sides at a point front of the rear axle sufficient to throw a proper proportion of the load on the front axle; this brings the center of gravity about one foot back of a point midway between the axles, which, practice has demonstrated, is the proper distribution of a load over the two axles to secure the easiest draft with wheels of the relative height of those usually used.

The springs are made with a double sweep so, as the weight in the body causes them to settle, the bearing on the springs comes nearer to the center of the springs, thereby strengthening the springs and practically making a graduated spring which automatically is adapted to the weight to be carried.

Attached to the front end of the body is a chain which passes through the top blocking and gear under the front axle; this chain passing under the axles in a measure takes off the strain or jar caused by dumping the body, the front springs acting as a cushion, and the body is prevented from striking the ground at the rear end. This admits of the body being drawn from under a load which is partially on the ground without tearing away the body or straining the horses. This chain can be lengthened or shortened at will, thereby determining the angle of body when tilted which will need to vary to suit different conditions, as nature of load, place where load is to be dumped, etc.

Attention is called to the device for holding the tail board in place. This is a bell-cranked hook, the upper portion of which is connected by a chain to the lower frame; as the body is tilted the distance between the back end of the body and the frame increases, thereby tightening the chain, which, when a given distance is traversed by the body, automatically pulls the hook out of the eye in tail board, and, the tail board being released, the weight of the load causes it to open and allow the load to pass out. With this arrangement the driver can unload his wagon without getting off it.

FIG. 38. VIEW SHOWING METHOD OF EXCAV ATING TRENCH IN ROCK ALONG EAST SIDE OF UNION SQUARE.

Throughout this section, as was the case elsewhere, special provision had to be made for handling the subsurface pipes during construction. As the excavation was opened the pipes encountered were suspended by chains from cross timbers. When their depth and position were such as not to interfere with the subway structure they were simply packed around with earth during backfilling and not disturbed. In other cases they were raised, lowered or rebuilt, as was most convenient in each case. Fig. 40 is a view showing the method of lowering a 36-in. water main to a level about 5 ft. below its original position. As will be seen, timbers were placed across the trench, through each of which two screw bolts were passed which carried at their bottoms a yoke or saddle, on which the pipe rested. When all the yokes were in place the ground beneath the pipe was excavated and the trench carried down 5 ft. deeper while the pipe hung suspended on the yokes. When the new foundation bed was ready the pipe was lowered in a few hours by a gradual [illegible] taneous loosening of the screw bolts.

The work of constructing the subway st[illegible] calls for only brief mention. Its chief diff[illegible] arose from the necessity of working in c[illegible] quarters underground. The general mode [illegible] cedure was to place the concrete floor of t[illegible] way and set the stone footing blocks, and [illegible] erect and rivet up the steel bents. The r[illegible]

was done almost entirely with pneumatic percussion riveters, which were found on trial to be decidedly more economical and to do better work than hand riveters. In passing mention should be made of the use of quick stroke pneumatic hammers fitted with drill bits for drilling the anchor bolt holes in the column footings, which resulted in a large saving in time and labor. As soon as the steelwork was in place the jack arches of the side walls were carried up in wooden forms and the roof arches placed on wooden centers. The concrete was usually mixed by hand on the floor of the subway.

The work described was carried on from 6.30 a. m. to 11 p. m., using two eight-hour shifts, with a 30-minute interval between them. For night work light was supplied by Kitson incandensecnt lamps. This lamp consists of an oil vaporizer and an air tube leading to and supporting a burner having a mantle similar to the familiar Welsbach

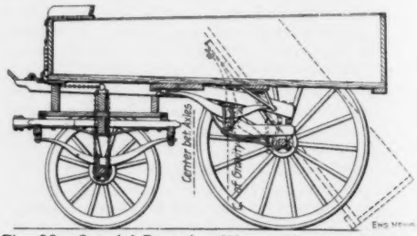


Fig. 39. Special Dumping Wagon Used on Contract Section III.
Shadbolt Mfg. Co., Brooklyn, N. Y., Builders.

gas mantle. At the beginning of the work some trouble was had with the labor unions by trying to erect the steelwork with non-union labor, but the dispute was compromised.

POWER PLANT.—The power for operating all machinery on Contract Section III. is transmitted by compressed air from a central power plant at the northeast corner of Union Square. At this point permission was secured by the contractors

to enclose an area of the park about 100 ft. square and adjacent to the excavation. On this track were erected a frame structure containing a machine shop, blacksmith shop, carpenter shop and store room. Adjacent to this building a larger frame building was erected to serve as a boiler and engine house. There are five 100-HP. fire tube boilers arranged in two batteries of two and three boilers, respectively. These boilers supplied steam to three 24½ × 30-in. straight line Ingersoll-Sergeant air compressors, pumping to a 4½ × 12-ft. receiver. From the receiver an 8-in. delivery pipe connects with the supply pipe, extending north and south along the work about 4,000 ft. each way. The supply pipe is 5 ins. in diameter from the power house, 2,000 ft. each way, and then decreases to 4 ins. in diameter. Besides the main machines just mentioned, the power house contains an ample equipment of boiler feed pumps, feed-water tanks, feed-water heaters, etc. The air was supplied to all machinery working on the line, no direct steam power being used. It should also be noted that the contractors have installed a stone crusher for working up the excavated rock into material for concrete.

Before compressed air was adopted as a motive power for the subway work, the contractors made a careful study as to the availability of steam and electricity. Although steam appeared the most economical, it necessitated many isolated boilers and engines, with endless dirt and confusion, besides the disadvantage of working many union engineers. Electricity appeared economical, but there was no assurance of satisfaction in electric drills. An electric power plant would have proved an expensive installation, and the cost of electric power from local companies was about double the estimated cost of compressed air.

The adoption of compressed air on this section was quickly followed by the installation of similar plants for other sections of the subway, and it has proved even more satisfactory and economical than was anticipated. The principal difficulties have been on account of condensation of moisture and the expansion and contraction of the pipes, as well as the freezing at exhausts in cold weather. The condensation was provided for by a small drip valve in the bottom of the receiver, and an additional drip and blow-off at the low point of the pipe line. The latter is blown off before starting work every morning. Air heaters were made and installed at each machine; these prevent freezing at the exhaust, and are considered economical even in warm weather. Provision for expansion was made by adoption of expansion joints, 500 ft. apart.

Tests of the power plant show the following efficiency:

Average steam pressure, lbs.	96
Average air pressure, lbs.	98
Water evaporated per lb. of coal (212°), lbs.	8.92
Coal per 1,000 cu. ft. of free air, lbs.	14
Average temperature of boiler feed, degs. F.	170
Average temperature air entering receiver, degs. F.	300

DESIGN OF A CONCRETE-STEEL ARCH BRIDGE.

By Daniel B. Luten.*

The design of an arch bridge is usually so dependent upon indeterminate external forces and upon doubtful assumptions as to internal stresses that an efficient structure is practically an impossibility if built according to any one of the usual methods of designing such structures. "Many masonry arches are excessively strong, and hence there are empirical formulas which agree with existing structures, but which differ from each other 300 or 400%."† Arches have been designed and erected according to one theory and series of assumptions, and other arches under similar conditions have been designed according to other theories and assumptions, but requiring not more than one-half the material used in the former, and yet the arches designed according to either method stand and show no indications of failure.

This condition is by no means unusual in arch design at the present time. It is especially true of the design of arches of concrete. A railroad with headquarters at Chicago builds a 20-ft. concrete arch culvert with a thickness at the crown of 36 ins.; another railroad, similarly located and under similar conditions, builds a 20-ft. arch with an 18-in. crown, and other parts in proportion.

Large trunk sewers are almost invariably built of lighter section than culverts of the same opening, although the culvert is usually designed with substantial foundations and abutments such as are rarely provided for the sewer. In an Indiana county the engineer approves the use of a 6-ft. steel culvert consisting of a cylinder of ¼-in. steel, laid on a gravel bed, and covered to a depth of 15 ft. with earth, and the same engineer designs a semicircular arch of concrete of 8-ft. span to support 3 ft. of fill, with a thickness of 2 ft. at the crown and 4 ft. at the springing. Some of these examples are accounted for in part by deficient training and experience in the engineers in charge, but even the most experienced engineers in arch design produce results that are remarkable for their lack of agreement.

Westminster Bridge, in London, is of granite semicircular arches of 76-ft. span, having a thickness at the crown of 7 ft. 7 ins. and at the springing of 14 ft. Compare this with the Conemaugh Viaduct, on the Pennsylvania Ry., of sandstone, span 80 ft., semicircular, with a thickness at the crown of 3 ft. and at the springing of 3 ft. 6 ins.

The difficulty in securing efficiency of design in an arch is owing to the impossibility of determining (1) the direction of pressure of the earth filling over the arch and back of the abutments, (2) the amount of resistance offered by the abutments to the horizontal thrust of the arch ring, and (3) the stability of earth foundations subjected to varying pressures at the base of the abutment. And after safe values have been assumed for the external forces, there still remains the analysis of the stresses produced in the arch ring and abutments, which can be determined only by making doubtful assumptions which require large factors of safety.

During the past year the writer has designed and erected 11 concrete-steel arch bridges in Indiana, two in Michigan and three in California, according to a method of design which, it is thought, eliminates some of these doubtful factors. The method is explained in detail in the following description of the design and erection of one of these bridges, on Pike St., in the city of Pontiac, Mich. This bridge was built in August, 1901, to replace a wooden bridge that had been declared unsafe. The span required was 30 ft., the width from property line to property line 60 ft., and the height from bed of stream to surface of roadway 9 ft. The depth of water in the stream was ordinarily not more than 18 ins.

The method followed in designing the arch is based on the principle that the most economical form of arch ring is a linear arch for the given loading, with thickness of rib proportional to the thrust. In such an arch the stress would be uniform, every particle of material being subjected to compression of the same intensity, with consequent economy of material. The direction of the earth pressures cannot be determined, but the limiting positions for the two extreme conditions of earth loading are known: (1) the earth pressures may be vertical, a condition that might be realized in rare cases of very compact, tenacious earth loading; or (2) the pressures may be normal to the outer surface or extrados of the arch ring and abutments, a condition of fluid pressure that might obtain when the loose fill over the arch is thoroughly saturated and semi-fluids. Between these two extremes must lie every condition of earth loading, except that produced by ramming the earth fill securely behind the abutments and haunches; tamped earth would by slightly distorting the arch, act against it with a greater horizontal component than that due to fluid pressure, but such pressure could not be active and would be in the nature of a reaction only, enabling the abutments to better resist the horizontal thrust of the arch due to live load, and thus adding a factor of safety to the structure.

If the pressures of the filling on the arch ring be assumed as everywhere vertical and proportional to the column of earth directly above, the curve of equilibrium will be the catenary, or its vertical projection the transformed catenary; if the pressures be assumed as normal to the outer surface of the arch ring, the curve of equilibrium will be an approximation to the hydrostatic arch. If the arch rib be assumed to be hinged at the crown, these two curves of equilibrium may be determined so as to have a common point of tangency at the crown, diverging as they approach the abutments, the curve for vertical pressures lying above the other; the curve of equilibrium for any condition of the earth loading whatever must then lie between these two curves. The curve of equilibrium for vertical pressures must be determined first, and the upper bounding surface of the arch rib be chosen to conform to this curve of equilibrium. The fluid pressures are then assumed normal to the extrados so defined, the resulting curve of equilibrium drawn, and the interior bounding surface or intrados of the arch rib chosen to conform to this latter curve. In determining the dimensions of the arch rib, no attempt is made to confine the curves of equilibrium to the middle third of the rib; in fact, the extreme positions of the curves will, in general, lie without the middle third, yet, as will be shown

FIG. 40. VIEW SHOWING METHOD OF LOWERING 36-IN. WATER MAIN.

*La Fayette, Ind.
†Baker's "Masonry Construction."

later, without endangering the stability of the arch ring.

No attempt is made, either, to make the curves of equilibrium as determined in this manner agree with a strictly mathematical determination of the curves for the elastic arch. All variations, however, are on the side of safety. The method offers an easy solution, and, at least for small arches, results in an efficient design.

The method of procedure is shown in detail in Figs. 1 to 4. On the left in Fig. 1 is determined the curve of equilibrium for the vertical pressures, and on the right is shown the resulting arch rib, for vertical pressures only, the thickness of the rib varying with the thrust. The segment of the desired curve is to have a span of 30 ft. The depth of earth filling over the crown is to be 18 ins., and the thickness of the arch rib at the crown is tentatively assumed as 12 ins. The segment must have sufficient rise in the given span to afford ample waterway, and to give an artistic design for the finished structure; this rise is assumed in this case to be 3 ft. 6 ins. These assumptions determine three points in the curve. The curve of equilibrium for vertical loads, bounded by a horizontal line above and by the curve below, is the catenary, or its vertical projection the transformed catenary, determined by increasing or decreasing each of its ordinates from the horizontal line in the same proportion. Since the catenary cannot, in general, be made to pass through three points, it is usually a transformed catenary which is the desired curve of equilibrium. The method of properly choosing the catenary that can be projected into a segment passing through the three given points, and the mode of drawing the curves, will be explained in detail later.

In Fig. 1, b c is the catenary from which is derived the transformed catenary b' c', which is the curve of equilibrium for the given vertical loading, and passes through the three given points. This transformed catenary is the center line of the arch rib of Fig. 1, which is assumed to be 12 ins. thick at the crown and increases in thickness with the secant of the angle of inclination of the rib. In proportioning the thickness of the rib according to the thrust, we have, however, disregarded one of our assumptions; the upper and lower surfaces of the concrete rib, in order to give the proper loading, should be transformed catenaries derived from the same original catenary. This would increase the thickness of the rib slightly, but as the weight of the gravel filling is very nearly the same as that of the concrete, this variation in the curve of the extrados is immaterial; and since the lower curve, even if properly drawn as a transformed catenary, would fall within the curve of the intrados as finally determined, the variation is on the side of safety. The arch rib is shown on the right, together with the earth filling, the curves of the intrados and extrados being drawn as arcs of circles with radii decreasing rapidly towards the abutments. The horizontal thrust in the rib under the earth loading alone is found from the force polygon to be 13,100 lbs. per foot of width, assuming that the weight of the earth filling is 125 lbs. per cu. ft. and that of the concrete 140 lbs. per cu. ft. The resulting intensity of pressure on a section at the crown is 91 lbs. per sq. in.

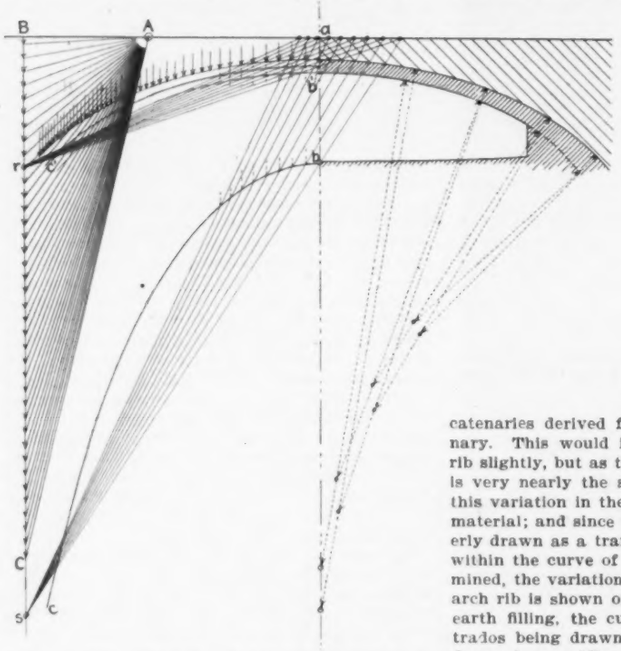


Fig. 1. Determination of Arch Rib for Vertical Earth Pressure. 80-ft. Concrete-Steel Arch Bridge at Pontiac, Mich.

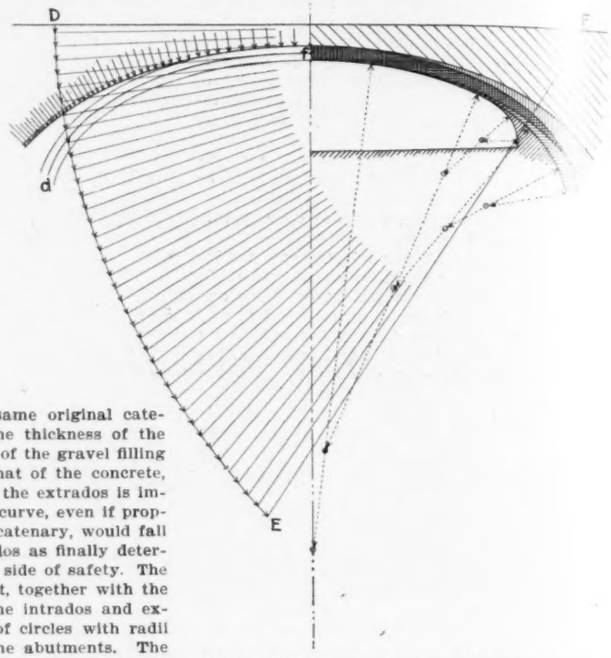


Fig. 2. Determination of Arch Rib for Normal Pressures and Combination of the Results.

In Fig. 2 this curve of the extrados is reproduced with the pressures shown as now assumed normal to the curve. Assuming the same horizontal thrust as that of the transformed catenary, the new curve of equilibrium is determined from the force polygon for the normal loading. It is the curve f d, and since the thrust along this linear arch is practically uniform, the [illegible] of the intrados and extrados for this rib a[illegible]own parallel to f d.

On the right of Fig. 2 are combined th[illegible]ults of the operations of Figs. 1 and 2. The [illegible] rib represented with vertical shading is t[illegible]uilibrated rib for vertical loading; the rib [illegible]d horizontally is the equilibrated rib for ea[illegible]ress-sures normal to the extrados of the for[illegible]rib. The first rib would have uniform stress [illegible]sh-out for vertical pressures produced by [illegible]act earth loading, which condition would obta[illegible]re and more as the arch became older. The lower rib would have uniform stress throughout if the earth pressure were that of a fluid of the same density as the earth filling and resting upon the former arch rib.

If, now, the extrados of the upper rib and the intrados of the lower rib be taken as the extrados and intrados respectively of a new rib hinged at the crown, the curve of equilibrium for any condition of earth loading will lie within the central axes of the two original ribs. If material be added eccentrically to a rib under uniform compressive stress, it can be shown that such an addition may increase the intensity of compressive stress by one-third as a maximum, or may produce tension in part of the material to an amount not exceeding one-third of the original compressive stress. Having determined that the uniform compressive stress in either rib would be 91 lbs. per sq. in. for the earth loading, the maximum intensities of stress in the new rib may be determined by assuming that the curves of equilibrium occupy either of the extreme positions. The intensity of compressive stress cannot, then, at any time be greater than 1⅓ times 91, or 121 lbs. per sq. in., and the tensile stress cannot be greater than one-third of 91, or 30 lbs. per sq. in. These stresses are barely one-half the allowable stresses for concrete, which in the specifications for this bridge were limited to 300 lbs. per sq. in. for compression and 60 lbs. per sq. in. for tension. (And 100 lbs. per sq. in. for shear; but the equilibrium curve follows the central axis so closely for this loading that the shearing stresses are practically nothing.) An examination of the force polygons shows that the arch will safely support an additional distributed load increasing from 300 lbs. per sq. ft. at the crown to 800 lbs. per sq. ft. at the haunches, if this additional loading be properly distributed over the surface of the roadway.

The curve of the intrados as finally adopted for the arch was slightly modified by the addition

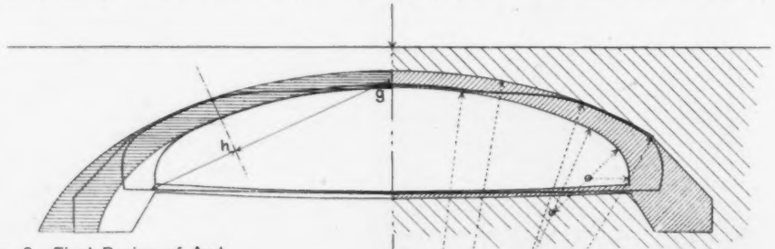


Fig. 3. Final Design of Arch.

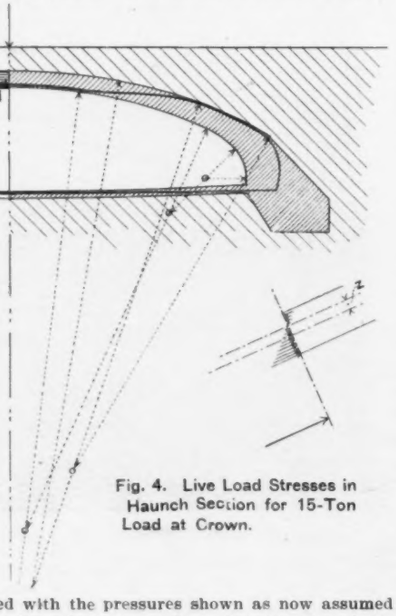


Fig. 4. Live Load Stresses in Haunch Section for 15-Ton Load at Crown.

of material as shown by diagonal shading on the right of Fig. 2. The curve is a seven-centered curve having a span of 30 ft. and a rise from springing to crown of 5 ft. 6 ins.

In arches as ordinarily designed, the horizontal thrust of the ring is resisted by an abutment consisting of a heavy mass of masonry, whose great weight prevents overturning or slipping on the foundation. Such a device, consisting of material the most of which is subjected to but slight stress, of 3,200 lbs. A section through the rib and normal to it is shown to an enlarged scale in Fig. 4. The forces acting on this section are the thrust of 3,200 lbs. and the thrust in the arch rib caused by the earth loading, which is assumed at its extreme lower position in order to give the worst case, and is found from the force polygon to be 13,500 lbs. per foot of width of the bridge. The moment of the resultant of these two forces is balanced by the moment of resistance of the of analysis, when applied at various points in the rib, determines the size of the steel rods in the arch rib. A stress of 10,000 lbs. per sq. in. requires ¾-in. rods spaced 2 to 4 ft. apart. The tie-rods in the pavement below, spaced at the same intervals, are 1 in. in diameter.

On account of the complexity of the problem, no attempt is made to analyze the stresses in this arch that would be produced by expansion or contraction due to changes of temperature. It may

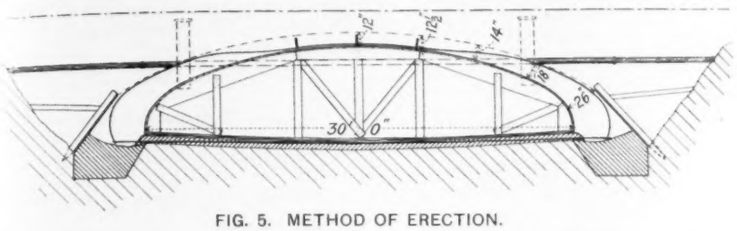


FIG. 5. METHOD OF ERECTION.

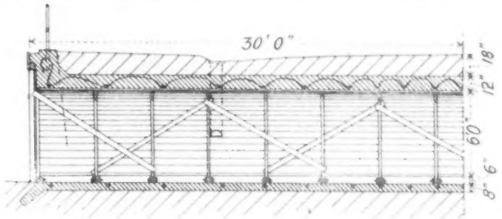


FIG. 6. HALF-TRANSVERSE SECTION AT CROWN.

is manifestly uneconomical, its purpose being not so much to resist the thrust as to divert it. In the Pontiac Bridge the thrust is resisted by tie-rods from abutment to abutment beneath the bed of the stream. The ties are of steel, embedded in a layer of concrete 8 ins. thick, which forms a pavement with its surface flush with the bed. The expense of laying this pavement is more than offset by the saving in weight of the abutments, which are proportioned as a part of the arch ring and not intended by their weight to resist any great proportion of the horizontal thrust. Besides the saving in material of the abutments, this pavement adds other advantages to this type of structure: it makes the bridge absolutely flood-proof by preventing scour of the stream and undermining of the abutments; it provides a substantial foundation for the forms supporting the arch ring during erection, eliminating a large part of the danger of distortion of the arch ring before hardening, a defect that might prove fatal in an arch of concrete; and it confines the earth of the foundations, thus helping to prevent settlement of the abutments in soft foundations. The use of the tie transforms the horizontal thrust of the arch into a vertical force equal to one-half the weight of the loaded arch; with proper proportions for the base of the abutments, the pressure on the base will be of uniform intensity and is easily determined. The form of abutment selected was slightly modified from the theoretical rib for practical reasons, and, together with the pavement and tie, it is shown in Fig. 3. The pavement is depressed 6 ins. at the middle of the stream, and the rod is embedded to a depth of at least 3 ins. The ties are not necessarily straight, but may also be depressed at the middle of the stream, provided they are not depressed so much that the pull in the rods would lift the pavement. The limiting amount of depression allowable in the ties of the arch here considered is computed to be 8½ ins. The rods are proportioned to resist the entire horizontal thrust due to dead and live loads, but as the major part of the load is steady load and a large but indeterminate portion of the thrust is resisted by the earth backing, the steel is figured at its ultimate strength.

The stresses produced in the rib by the live load can only be determined approximately. If we assume a load of 15 tons, as required by the specifications, to be concentrated at the crown and transmitted by the earth filling to 10 ft. of arch rib—that is to say, 3,000 lbs. per ft. of width of roadway—one-half of this load would be transmitted to each abutment, producing a vertical live load reaction of 1,500 lbs. per foot of width. In order to be on the safe side, it is wise to assume that the abutment is not fixed, but free to rotate, as would, indeed, be the case if the foundations should yield ever so little. The point about which rotation would take place would be the junction of the tie-rod with the abutment. Assuming, also, to simplify matters, that the arch rib is hinged at the crown, the line of thrust due to the concentrated load would be the straight line g h joining crown and abutment. The amount of this thrust would be the vertical force of 1,500 lbs. resolved upon its line of action, giving on this line a force stresses in the rib. Making the usual assumption that the stresses vary uniformly as their distance from the neutral axis, the maximum compressive stress on the section would be 287 lbs. per sq. in., and the maximum tensile stress 132 lbs. per sq. in. As the earth loading on the arch becomes more compact, the equilibrium curve would recede from its extreme position, reducing these stresses somewhat. The compressive stress is well within the safe limit specified, but the tensile stress is excessive, and the concrete is consequently reinforced with steel rods, located, as shown in Fig. 3, with their ends joined to the lower tie-rods in the abutments. The rods pass close to the tension edge of the arch ring, being placed as nearly as possible in the line of the resultant tension at its maximum.

It is not to be supposed that the above is an entirely satisfactory method of analysis for a concrete and steel combination. In the present state of our knowledge concerning these materials, it is perhaps as satisfactory as any, and, judging by recent experiments, it certainly gives results that have a large margin of safety.

The dimensions of the arch ring have been determined upon the assumption that the rib was hinged at the crown. As a matter of fact, the arch in erection was not hinged at the crown, but was made continuous, although in a larger structure it would probably be advisable to supply the hinge. The omission of the hinge results in a bending moment at the crown, tending to relieve the moments at other points. This moment at the crown would, for the concentrated load, produce tension along the intrados near the crown. The steel rods are therefore carried along the lower edge of the rib at the crown, as shown in the

Fig. 7. View of Completed Bridge.

figure, as would, indeed, be done even if the arch were provided with a hinge at the crown. The bending moment at the crown is indeterminate, so that it is impossible to say whether or not the proportions of the rib at that point are adapted to resist the stresses produced. It is certain, however, that if the tensile stresses are excessive and should cause a crack in the concrete at the crown, the arch would then behave as a hinged arch, and will conform to our analysis. The above method be pointed out, however, that the rods are embedded in the arch rib in a position that enables them to resist the tendency to distortion resulting from a change of temperature. With decreasing temperature, the arch ring is shortened, the abutments remaining fixed, and the crown of the arch is depressed, producing tension in the material at the lower side of the crown and at the upper side of the haunches. But steel has a higher coefficient of expansion than concrete by about 20%; with falling temperature, the steel will contract more than the concrete, and will tend to compress it at the lower side of the crown and over the haunches, thus counteracting the tendency to distortion and probably reducing the stresses by distributing them. With increase of temperature the same conclusion holds.

It may be wise to repeat before taking up the solution in detail that the above analysis is illogical, in that the stresses are determined by assuming that the material is distorted, while the equilibrium curves are found on the assumption that the rib is inelastic. An attempt to analyze the rib as elastic shows, however, that the equilibrium curves would have to approach the central axis to conform to the conditions required by an elastic rib, and that therefore the above analysis is a safe one.

The bridge was erected complete in 28 days and immediately thrown open to traffic. The foundations for the abutments were first placed, as shown in Fig. 5, and the pavement was then laid in two sections, it being necessary to cofferdam to exclude one-half of the stream at a time. At the time the pavement was laid the water was backed up by the closing of a mill below, so that it was necessary to dam back 3 ft. of water to lay the concrete. In laying the first half of the pavement, the rods were bent up to clear the cofferdam which extended along the center line of the stream; then, when the dam had been moved over to enclose the other section of the pavement the rods were bent down and embedded.

After the pavement had hardened for two days, the forms were erected upon a raft of 2-in. planks, which sank with the weight of the drum, to a bearing on the pavement, as shown in Fig. 5, and wedges were driven under each of the uprights. The centers consisted of a single thickness of 2-in. planks spaced 4 ft. apart, the curved pieces being shaped at the mill. The forms were thoroughly saturated with water and kept so throughout the construction, to prevent shrinkage of the forms. The concrete of pavement and arch ring was uniform throughout, consisting of Medusa brand of Sandusky cement and unscreened gravel in the specified proportions of 1 to 3 to 5, or the equivalent 1 to 7. The steel rods were smooth, round, of medium steel, with scarf welds and open hooks turned at the ends, which were upset to prevent pulling out of the concrete. Since these rods are in tension throughout, the writer does not consider that there is any necessity for corrugations or projections on the rods to distribute pressures, and practice seems to confirm this view. The arch ring was built in annular sections, extending from abutment to abutment. A ring 16 or 18 ft. in width would be completed

in one day by two gangs of men mixing, one gang on each bank of the stream, and each gang completing one-half of the rib. All exposed surfaces were finished with a 1-in. layer of 1 to 2 cement mortar, tamped into place with the concrete to form a solid whole.

The forms were removed about 30 days after the filling and roadway had been completed and thrown open to travel. Most arches, when the centers are struck, deflect at the crown, due probably not so much to excessive flatness at that point as to slight yielding of the abutments. In the 16 concrete-steel bridges erected during the past year, the fill and roadway over all but three were completed before the forms were removed; in all but one the crown of the arch was found to have risen free from the centers before striking, a condition that might have been due to shrinkage of the forms. But in none of these cases were there any indications whatever of any settlement at the crown.

A 4-in. gas pipe traverses the arch ring, as shown in the protograph. The bridge rails were of 2-in. gas pipe with posts embedded 1 ft. in the parapet wall, this type being chosen because of cheapness. A view of the completed bridge and surroundings is shown in Fig. 7.

The cost of the bridge complete, including walks and roadway of gravel, was $2,280, which is not more than a steel bridge with wooden floor of half the carrying capacity would have cost.

METHOD OF CALCULATION.—Having determined by trial that a catenarian rib of 30-ft. span and 3-ft. 6-in. rise will answer the conditions of necessary waterway and desired appearance, it is required to determine a catenary which will be in equilibrium under loading represented by its ordinates to the directrix, and which can be vertically projected into a transformed catenary of the required dimensions. The equation of the catenary is

$$y = \frac{m}{2}\left(e^{\frac{x}{m}} + e^{-\frac{x}{m}}\right).$$

If the ratio in which the ordinates are decreased in order to produce the transformed catenary be represented by r, then the ordinate of the latter curve y^1 is equal to

$$y^1 = r\,y = \frac{r\,m}{2}\left(e^{\frac{x}{m}} + e^{-\frac{x}{m}}\right).$$

But r m is equal to the ordinate at the crown of the transformed catenary. Calling this ordinate m^1 we have for the equation of the transformed catenary,

$$y^1 = \frac{m^1}{2}\left(e^{\frac{x}{m}} + e^{-\frac{x}{m}}\right).$$

Solving this equation for m we shall have

$$m = \frac{x}{\text{hyp. log.}\left(\frac{y^1}{m^1} + \sqrt{\left(\frac{y^1}{m^1}\right)^2 - 1}\right)}$$

which determines the ordinate to the crown of the catenary when the crown and one other point of the transformed cantenary are known.

In the case under consideration n^1 equals 2 ft. 0-in., x equals 15 ft. 0-in., and y^1 equals 5 ft. 6 ins. From which m is found to be

$$\frac{15}{\text{hyp. log. } 5.312}$$

As tables of hyperbolic logarithms are comparatively rare, the following table for transforming from common to hyperbolic logarithms is given:

Logarithms— Common.	Hyperbolic.	Logarithms— Common.	Hyperbolic.
1.	2.302585	6.	13.815510
2.	4.605170	7.	16.118096
3.	6.907755	8.	18.420681
4.	9.210340	9.	20.723266
5.	11.512925	10.	23.025851

The common logarithm of 5.312 is 0.72526; adding together the numbers in the above table, which correspond to the figures of the common logarithm, we have the hyperbolic logarithm. Thus:

0.7	1.6118096
2	40052
5	11513
2	400
6	138
	1.668261

Hyperbolic logarithm of 5.312 equals 1.67, and m equals 9 ft. 0-in. This distance laid off vertically downwards from a locates the vertex b, Fig. 1, of the required catenary. It also represents the horizontal thrust A B of the force polygon provided the divisions of the load line be made equal to those of the arc of the catenary. The load line BC is divided into any convenient number of equal parts commencing with a half-division at B. The elements of the curve are then drawn parallel to successive rays of the force polygon, each element being made equal to the divisions of the load line. The transformed catenary is derived by reducing each ordinate of the catenary in the ratio of nine to two. This is easily done, graphically by assuming two points r and s on any convenient vertical line, and at distances from the directrix in the ratio of two to nine, and then passing lines from s through points on the catenary to meet the directrix, and from these points on the directrix back to r. It will readily be seen that such lines will divide the vertical ordinates in the same ratio of two to nine, and will consequently locate points of the transformed catenary. The horizontal thrust is the same for the transformed catenary as for the catenary, the load in each case being limited by the curve. The thrust in the rib, as will be seen from the force polygon, varies as the secant of the angle of inclination with the horizontal.

In Fig. 2, by spacing the points of application of the normal loads at the same horizontal intervals as the vertical loads of Fig. 1, the normal pressures will be equal, and the load line may then be drawn with elements of uniform length parallel to successive normal loads. In determining the normal loads the weight of the arch rib, which should be taken as vertical, is assumed to be in the same direction as the normal pressure acting at the point; this simplifies the process and results in deflecting the curve downwards, thus counteracting the error of assuming the thrust in the rib to be uniform, when in reality it increases slightly towards the abutments, as may be seen in the force polygon. The elements of the curve of equilibrium are drawn parallel to the successive rays of the force polygon, and limited in length by the points of intersection with the normal forces.

If the center line of an arch rib is a linear arch for the given loading, the resultant of the stresses on any normal section will pass through the center of gravity of the section, and the intensity of stress will be uniform over the area of the section. If now material be added at one side only of the section, the position of the resultant to be unchanged, the intensity of stress on the opposite side of the section will be increased, while on the side to which the material is added it will be diminished. In the present case we have to deal with a rectangular section. Assuming the thickness of the rib originally to be 2a, and after the addition of material to be 2x, the distance between the central axis and the neutral axis to be z, f the intensity of stress in the extreme fiber, and F the resultant force acting through the center of gravity of the original section, the stress to be considered as increasing uniformly with its distance from the neutral axis, we have by taking moments about the center of gravity of the tensile stresses on the section,

$$F\left[(x - a) + z + \tfrac{2}{3}(x - z)\right] = f\frac{(x+z)}{2}\left[\tfrac{2}{3}(x+z) + \tfrac{2}{3}(x-z)\right].$$

And by summation of forces on the section,

$$F = f\frac{(x+z)}{2} - f\frac{(x-z)}{x+z}\frac{(x-z)}{2}.$$

From these two equations may be derived,

$$z = \frac{x^2}{3(x-a)}.$$

Also,

$$f = \frac{F}{2}\frac{z+x}{z\,x}$$

for the compressive stress in the extreme fiber, and

$$f^1 = \frac{F}{2}\frac{z+x}{z\,x}\left(\frac{z-x}{z+x}\right)$$

for the tensile stress in the extreme fiber on the opposite side of the section. From which

$$f = \frac{F}{2}\frac{4x - 3a}{x^2}$$

and

$$f^1 = \frac{F}{2}\frac{2x - 3a}{x^2}.$$

The maximum values of f and f^1 are determined by placing their first differential coefficients with respect to x equal to zero,

$$\frac{df}{dx} = \frac{F}{2}\left[\frac{(4x - 3a)2x - 4x^2}{x^4}\right] = 0,$$

from which $x = \frac{3}{2}a$, and the resulting value of f is $\frac{4}{3}\frac{F}{2a}$. Also,

$$\frac{df^1}{dx} = \frac{F}{2}\left[\frac{(2x - 3a)2x - 2x^2}{x^4}\right] = 0,$$

from which x equals 3a and $f^1 = \frac{1}{3}\frac{F}{2a}$.

$\frac{F}{2a}$ is, however, the intensity of the uniform stress on the section before the addition of material at one side of the section. It follows, therefore, that the addition of material on one side of a rib under uniform compression, may increase the intensity of compressive stress by not more than one-third, and may produce tensile stress not greater than one-third of the uniform compressive stress.

The analysis for live load stresses is similar to the above. The forces acting on the section shown in Fig. 4, are the dead load thrust of 13,500 lbs. acting at a distance of 3 ins. from the central axis, and the live load thrust of 3,200 lbs. acting at 30 ins. The thickness of the rib at the point of section is 18 ins. Taking moments as before about the center of gravity of the tensile stresses,

$$\frac{13500}{12}\left[3 + z + \tfrac{2}{3}(9 - z)\right] + \frac{3200}{12}\left[30 + z + \tfrac{2}{3}(9 - z)\right] = f\left(\frac{9+z}{2}\right)\left[\tfrac{2}{3}(9+z) + \tfrac{2}{3}(9-z)\right];$$

and summing the forces on the section,

$$\frac{13500}{12} + \frac{3200}{12} = f\left(\frac{9+z}{2}\right) - f\left(\frac{9-z}{9+z}\right)\left(\frac{9-z}{2}\right).$$

from which f is found to be 287 lbs. per sq. in., the compressive stress on the extreme fiber. And the tensile stress is found by proportion from this value to be 132 lbs. per sq. in. Assuming that 60 lbs. of this stress is carried by the concrete there remains

$$72\left(\frac{9-z}{2}\right)12 = 2{,}450 \text{ lbs.}$$

for each foot of rib to be carried by the steel rods, which are consequently chosen as ¾-in. rods spaced 2 ft. apart. A similar analysis made for other points in the rib shows these dimensions to be sufficient to satisfy the requirements.

When the load in concentrated at a point at one side of the crown, tension will be produced on the underside of the rib near the point of loading, if near the crown. The same method of analysis will show that the rod is required near the intrados at the crown for such cases of loading, and as its location near the extrados at the crown is not essential for any condition of loading, the rods pass from their position near the extrados at the haunches, across the rib at a point where the tensile stresses are a minimum, to follow the intrados for a short distance near the crown.

In testing small models of this type of concrete steel arches, it was found that when the lower tie was severed and the arch failed by spreading of the abutments, these rods embedded near the intrados would tear through the concrete, tending to straighten under the tension; to prevent this the transverse straps of 1 × ¼-in. steel were introduced, passing under the rods and weaving up into the concrete between the rods, as shown in Fig. 6. Three of these straps were used in the Pontiac Bridge, as indicated in Fig. 5.